Risk and Reliability Assessment of Multiple Reservoir Water Supply Headworks Systems

By

Philip David Crawley
B.E. (Hons) MEngSc.

Thesis submitted for the degree of
Doctor of Philosophy
in
The University of Adelaide
(Faculty of Engineering)

March 1995

Awarded 1995

To my wife Allison

ACKNOWLEDGEMENTS

The author wishes to thank his supervisor, Dr. G. Dandy, Associate Professor in the Civil and Environmental Engineering Department of the University of Adelaide, for his many hours of patient listening, valuable discussion and constructive criticism.

Funding for the initial stages of this work was provided by the Land and Water Resources Research and Development Corporation and the Engineering and Water Supply Department of South Australia through the Partnership Research Program. The completion of this research project has been solely funded by the Engineering and Water Supply Department. Thanks is extended to Neil Killmier and Peter Cooper of the Engineering and Water Supply Department whose foresight saw the potential long term benefits both to the department and the wider community from this research work and so provided the necessary financial support. Particular thanks is also extended to Peter Hoey, former Manager, Operations Support Branch, for his willingness to 'sell' the project within the Engineering and Water Supply Department and without whose boldness the work would not have been undertaken.

For the term of the project, a committee took on a review and oversight role and provided valuable input and direction to the research work. Members of this committee included Graeme Dandy, Peter Hoey, Peter Manoel and David Bain. The members of this committee are thanked for their contributions towards the project. The particular contributions of Peter Manoel are specifically acknowledged and were sadly missed following his untimely death.

During the course of the project, input concerning the reliability of the Adelaide bulk water transfer system was provided by a number of personnel from within the Engineering and Water Supply Department including Lionel Rodrigues, Hanley Pullar, John Minney, Jim Braendler, Dave Kerry, Tony Soar, Andrew Jessup, Tim Chong, Dennis Steffanson, Brian Smith and Rob Triggs.

Additional input was also provided by Bill Hagger, Glenn Gallasch and Eric Hurell of the Electricity Trust of South Australia and Bob Jordan of Mayfield Pty. Ltd..

In addition, a great deal of support has been provided by members of the Operations Support Branch of the Engineering and Water Supply Department in particular Brian Tattersall, David Bain, Andrew Jessup and Phil Pfeiffer.

My fellow postgraduates are also acknowledged for their support, in particular John McPheat for his amusing company.

I am extremely grateful to my family and friends who have been patient and provided encouragement throughout the project. Finally, I wish to thank my wife, Allison, who stood by me throughout the research work and gave me the necessary support to keep going when the task was difficult and for her patient review of this thesis.

All these people, and many others who helped are thanked for the contribution they have made to this work.

Statement of Originality

This work contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference is made in the text.

I give consent to this copy of my thesis, when deposited in the University Library, being available for loan and photocopying.

SIGNED :.. ..... DATE : ..3/4/95...

Synopsis

A methodology is presented in this thesis for the examination of reliability-cost tradeoffs in the operation of multiple reservoir water supply headworks systems.

The methodology considers the four major influences on the reliability of a water supply headworks system. These are the variability of inflows, the variability of system demands, the reliability of the bulk water transfer system and the application of water restriction policies.

Inflows into the reservoirs of a water supply headworks system from catchment runoff vary from month to month and year to year. The use of a synthetic inflow data generation model enables the examination of inflow variability on reliability-cost tradeoffs in the operation of multiple reservoir systems, and forms the first component of the methodology.

Demand for water is affected by a range of factors. The variability of demand from a water supply headworks system will also influence the reliability-cost tradeoffs in the operation of the system. In many water supply systems, the variability in demand will be smaller than the variability in inflow and hence have less impact on these tradeoffs. The use of a synthetic demand data generation model enables the examination of demand variability on reliability-cost tradeoffs and forms the second component of the methodology.

In order to assess the reliability of a bulk water transfer system, critical components within the system need to be identified. Reliability attributes then need to be determined for these individual components and their impact on the overall bulk water transfer system to be assessed. A technique entitled the 'walking party' approach has been developed in this research to identify these critical components and obtain realistic estimates of their reliability attributes. Using the technique of frequency duration analysis, results from the application of the 'walking party' approach can be combined. Results from the application of frequency duration analysis are then used as input to a Monte Carlo failure simulation model to generate random failures of the bulk water transfer system. Using these random failures the impact of the reliability of the bulk water transfer system on reliability-cost tradeoffs in the operation of multiple reservoir headworks systems can be examined. The inclusion of the bulk water transfer system reliability forms the third component of the methodology.

Water restrictions are a common mechanism used by water supply authorities to reduce demand on a water supply headworks system during periods of water shortage. The use of water restrictions to reduce demand is not without cost. Using the approach proposed by Dandy [66], the economic costs associated with the imposition of water restrictions have been considered and the impact of the use of water restrictions has been included in the assessment of reliability-cost tradeoffs for water supply headworks systems. The inclusion of the economic impacts of a water restriction policy forms the fourth component of the methodology.

The methodology presented in this thesis has been applied to the Adelaide Water Supply Headworks System and reliability-cost tradeoffs examined for a range of operating rule sets. The results obtained for the Adelaide system indicate that significant savings in operating costs can be achieved with little reduction in system reliability through modifications to the current operating rules.

Contents

List of Figures

List of Tables

Chapter 1

Introduction

1.1 Reasons for this Research

Over the last decade, strategies adopted throughout the world to satisfy public water supply needs have changed markedly. These changes have been forced upon water supply authorities by increased concerns about public health aspects of drinking water, declining availability of primary supply sources, and the need to consider the environmental consequences of the construction of dams and reservoirs. Greater competition for limited resources is forcing these authorities to seek more efficient use of existing resources as an alternative to capacity expansion of their supply systems.

Within Australia, this trend has become apparent with urban water supply systems moving from a 'construction' to a 'management' phase. In the majority of capital cities, the cheapest sources of water have been developed and the construction of major new dams are now unattractive due to a general shortage of capital funds. A primary focus of Australian water supply authorities has been on managing the existing systems to achieve maximum efficiency. The use of demand management techniques including water pricing has underlined the need to consider efficient use of the valuable water resources.

Most major cities throughout the world have a water supply headworks system consisting of a complex network of reservoirs, pipelines, channels and pumping stations. The operation of these complex systems is based largely on experience, supplemented in some cases by the use of computer simulation models. Work in Australia by Perera and Codner [232] for the Melbourne system and Crawley and Dandy [46] for the Adelaide system has demonstrated that potential increases in efficiency can be achieved by the use of mathematical optimisation techniques for the development of system operating rules. Such operating rules include target storage levels and target pumping levels.

A workshop on risk and reliability in water resources planning and management held in Adelaide, Australia in November 1988, recommended that 'risk management decisions of water authorities should be made with a level of community input appropriate to the risk situation' [61]. A further recommendation was that the 'Australian Water Resources Council (AWRC) should encourage water authorities to decide on risk offering, to the extent that is practical, a range of alternatives in security of supply and associated water prices so that consumers are in a better position to advise/select reliability they are prepared to pay for'.

In order to achieve these recommendations it is necessary to establish the trade-offs between reliability and operating costs. For water supply systems where a significant component of the water supply is transferred from a distant water source, not only does the variability of the natural inflows to the reservoirs need to be considered but also the reliability of the bulk water transfer system. It is evident that the larger the proportion of water obtained by transfer, the larger the likely impact of the reliability of the bulk water transfer system on the overall risk-cost tradeoffs for the water supply headworks system.

The research presented in this thesis addresses the major issues associated with the tradeoffs between reliability of a water supply headworks system and the economic costs associated with its operation. A methodology is presented which provides a total risk assessment for a water supply headworks system. The methodology integrates the major factors affecting the tradeoffs between system reliability and operating costs, with a particular emphasis on systems where a significant portion of the water supply is obtained by transfer from a distant water source.

1.2 Objectives of this Research Work

The objectives of this research work are :

- To identify the major factors affecting the reliability of urban headworks systems.
- To develop a methodology for determining the the tradeoffs between reliability and cost in the operation of these systems.
- To apply the methodology to a major water supply headworks system, in which transfer from a distant water source comprises a significant proportion of the total water supply.

1.3 Method of Approach

The consideration of risk and reliability issues is necessary in many fields of study. It is helpful to consider the wider risk and reliability definitions and analysis techniques before focusing on the specific problems at hand. The application of risk assessment and management to water supply systems is first placed in the broader context of the issues of risk and reliability in Chapter 2.

In order to examine the performance of a water supply headworks system, it is necessary to have a simulation model that accurately emulates the behaviour and operation of the system. The simulation model should be capable of accurately modelling alternative operating rules. Factors influencing the selection and development of an appropriate water supply headworks simulation model are presented in Chapter 3. The historical inflow and demand records available to simulate the operation of a water supply headworks system, are normally limited in length in relation to assessing the required reliability levels for the system. To test the performance of a water supply headworks system under conditions more extreme than those contained in the historical data set, it is necessary to generate synthetic data. Factors influencing the selection and development of an appropriate synthetic data generation model are presented in Chapter 3.

In most water supply systems, water is transferred between components of the system or transferred from a distant source, with the use of a bulk water transfer system comprising pumps, pipelines, and other electrical and mechanical equipment. The overall reliability of the water supply system will be influenced by the reliability of this bulk water transfer system.

In order to assess the overall reliability of a bulk water transfer system, critical components of the system need to be identified, reliability attributes for these components determined and the individual component reliabilities combined. In many situations, critical components of a system are not easily identified and specific data concerning their reliability attributes are scarce or non-existent. A technique has been developed as part of this research work entitled the 'walking party' approach that can be used to identify critical components in a system and obtain realistic estimates for their reliability attributes. The 'walking party' approach involves the interview of a number of individuals familiar with the system, followed by the formation of a smaller group from among these individuals entitled the 'walking party'. This 'walking party' visits and physically walks through the system. During the visit a consensus is reached regarding the critical components and an estimate of the reliability attributes associated with these components.

Having obtained estimates for the reliability attributes of these critical components, the technique of frequency-duration analysis, taken from the power industry, can be used to combine the reliability attributes of these components. Using this analysis technique, the reliability attributes of components in both series and parallel can be combined to obtain the overall reliability attributes for the system in the form of a state transition matrix.

Randomly generated failure events using the information contained in the system state transition matrix, in conjunction with synthetically generated inflow and demand data, can be used to examine reliability-cost tradeoffs for a water supply headworks system.

The use of restriction policies to manage demand during periods of water shortage is an increasing practice throughout the world. The application of these restriction policies, however, is not without cost. The final component of the methodology involves the inclusion of the use of restriction policies and their associated costs in the examination of tradeoffs between reliability and cost in the operation of a water supply headworks system.

The methodology developed for the assessment of reliability cost tradeoffs for a water supply system is applied in a case study of the Adelaide Water Supply Headworks System in Chapter 4.

Results from the application to the Adelaide system are presented and critically appraised in Chapter 5.

Ultimately, the choice of level of system reliability and an associated restrictions policy will depend upon the public (and hence political) acceptability. The outcome of this thesis is to demonstrate a methodology which can be used to assist in this decision-making process.

1.4 Thesis Structure

This thesis is organised in the following manner :

Chapter 2 contains a background to the general concepts of risk and reliability. In particular, some of the more common terms associated with risk and reliability are described. These terms are examined with particular reference to water supply headworks systems and a literature review of relevant work in this area is presented. Finally the research work presented in this thesis is related to current research in the field.

Chapter 3 presents a simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems. The methodology seeks to include the major elements that affect the reliability-cost performance for a headworks system. These elements include the variability of inflows, the variability of system demands, the reliability of the bulk water transfer system and the impact of water restriction policies.

Chapter 4 commences with a description of the Adelaide Headworks System, and its historical and current operating rules. A simulation/optimisation model that is currently used to assist in the operation of the system is described, which has been used as a tool to test the application of the proposed methodology to the Adelaide system. Details of the synthetic inflow and demand models that have been used to generate data for the Adelaide system are presented. The assessment of component reliability for the Adelaide system is detailed using the 'walking party' approach and the results obtained from the assessment presented. The chapter concludes with a description of economic costs associated with the use of a water restriction policy in the operation of the Adelaide system.

Chapter 5 describes results obtained for the Adelaide Headworks System applying the simulation methodology described in Chapter 3. The second section in the chapter describes results obtained considering the hydrological variability associated with the system. The third section includes the impacts of the component reliabilities in the simulation of the system. The fourth section of the chapter describes the simulation results obtained including hydrological variability, the component reliabilities together with the economic costs associated with the imposition of water restrictions.

Chapter 6 draws conclusions and recommendations from the work presented in the thesis.

1.5 Summary

In this thesis, a methodology is presented for the assessment of reliability-cost tradeoffs for a multiple reservoir headworks system where the system includes the bulk transfer of water from a remote source. The methodology involves the four major factors affecting these tradeoffs. These factors are :

1. The variability of inflows.

2. The variability of system demands.

3. The reliability of the bulk water transfer system.

4. The application of a water restriction policy.

The methodology involves the use of a water supply headworks system simulation model, synthetic inflow and demand data generation models, a technique for the identification and assessment of the reliability attributes of critical components in a bulk water transfer system, the application of 'frequency duration analysis' to combine the reliability attributes for the critical components, the generation of random bulk water transfer system failures and the inclusion of the economic costs associated with the imposition of water restrictions.

The application of this methodology to the Adelaide Water Supply Headworks System is presented and the results obtained are discussed.

Chapter 2

Review of Risk Assessment

2.1 Introduction

The purpose of this chapter is the following :

1. to detail some of the terminology used in the area of risk and reliability research,
2. to provide an overview of some of the techniques currently in use in this area,
3. to review research in the areas of risk and reliability relating to the operation and management of water supply systems, and
4. to place the research presented in this thesis in context with previously published work.

2.2 Terminology

It is first useful to consider the terminology used in the field of risk and reliability research. Having examined these terms, an appropriate set of definitions can then be selected for use within this thesis.

2.2.1 The System

One definition for a system is (Simpson and Weiner, [276]) :

> "A group, set, or aggregate of things, natural or artificial, forming a connected or complex whole."

A *subsystem* comprises an element of the system that may on its own also form a system.

If we consider a system as comprising a set of subsystems, assemblies, sub-assemblies and components, then a hierarchy can be formulated depending on the system type and complexity. This hierarchy is interconnected in such a way that the system can perform a specific function when input is provided from sources such as the system operators.

Within the context of this thesis, a water supply 'system' has been considered to comprise five subsystems. These five subsystems are :

- The water source.
- The bulk water transmission, pumpage and treatment.
- The bulk water storage.

- The finished water storage.
- The water users.

Each of these subsystems are described in greater detail in Section 2.4.

2.2.2 Risk

There are many definitions of risk that have been used historically, depending primarily on the context in which it is placed. One definition of risk is (Gove, [116]):

> "The possibility of loss, injury, disadvantage or destruction."

In the field of insurance, two types of risks are recognized :

1. Dynamic or speculative risks

   An example of this type of risk would be the outbreak of war. These risks are difficult to measure and can result in either a profit or a loss.

2. Static or pure risks

   An example of this type of risk would be the failure of a pipe. These risks result in losses only and are measurable statistically.

In an engineering context, two characteristics are normally associated with risk. Firstly, the probability that some event will occur and secondly, the measure of the associated impact, if the event does occur.

Kalbfleisch [161] defined risk as "a measure of the probability and severity of an occurrence that is, in some way, harmful to man". In a similar manner

Petrakian et al. [234] defined risk as "a measure of the probability of occurrence of a potentially hazardous event and of the event's consequence to society". Kaplan and Garrick [163] defined risk as "the sum of uncertainty and damage, or the ratio of hazard to safeguards". The Institution of Professional Engineers, New Zealand [156] has adopted the following dual definition of risk :

> "The probability that a potential hazard will be realised and the probability of harm itself."

All four definitions include something of the consequences or damage of the occurrence of the event as part of the understanding of risk.

Risk is directly associated with the likelihood or possibility of harm occurring for a system and its users, and involves something of the consequences of the occurrence. In the same way that a hazard may result in the occurrence of an accident, so risk relates to the probability that the frequency, intensity and duration of a certain stimulus will be sufficient to convert the hazard from a potential state to an actual loss. Risk can be considered as the potential for a mishap and can be expressed in terms of hazard severity and hazard probability (Roland and Moriarty, [254]).

With reference to a water supply system, Shamir and Howard [269] described risk as a measure of the consequences, particularly economic, of supply shortfalls. They note that the 'risk' to the security of supply of the system at any point in time could be described in terms of the probability, degree and duration of restrictive measures that would need to be applied to the system.

Within this thesis, only static or pure risks have been considered with regard to water supply systems. 'Risk' relates exclusively to the security of the water supply system and the potential for the system to be unable to meet customers' demands for water.

2.2.3 Failure

Failure is defined as (Hanks and Wilkes [123]) :

> "The act or an instance of failing, non performance, breakdown, insufficiency or shortage, loss of strength, not reaching the required standard."

A system that does not perform as required is considered to have failed. A simple definition of failure can therefore be given as :

> "An event, or inoperable state, in which any part of a system does not perform as previously specified."

Roland [254] notes that the failure of a system is commonly associated with the reliability of the system and is closely related to the system performance. Harr [129] defines failure of a system as "the inability of the system to perform its intended function". All systems fail eventually, however, the time between failures is a measure of the reliability of the system. Mean time to failure (MTTF) for a system is a measure of the expected period of time that the system will be operational.

With reference to a water supply system, Hashimoto et al. [130] defined 'failure' as "the occurrence of unsatisfactory performance of the system under consideration".

In this thesis, failure of a water supply system is defined as the inability of the system to meet the consumers unrestricted demand for water. The imposition of water restrictions on consumers, is therefore considered as a failure of the system.

2.2.4 Reliability

One definition of reliable is (Hanks and Wilkes [123]) :

> "Able to be trusted; predictable or dependable."

The Institution of Professional Engineers, New Zealand [156] describe reliability as being concerned with the probability of future events based on past observations. Harr [129] defines reliability as the "likelihood of the adequate performance of the system for a specified period of time under a set of operating conditions".

With regard to water supply systems, Hashimoto et al. [130] define reliability as "the frequency or probability that a water resource system will be in a satisfactory state". Mukjerjee and Mansour [216] define reliability for a pumped storage reservoir as "the degree of consistency for the reservoir to supply water without failing to meet the demand over a given period". Simonovic et al. [275] define reliability of the operation of a reservoir as "the opposite of risk or the probability of a system being in a satisfactory state".

With reference to a water supply system, the acceptance of a level of reliability must be viewed within the context of possible costs, risks and associated social benefits. The determination of the confidence limits of reliability measures of a system are also important in the overall assessment of the system.

Within this thesis, Hashimoto's definition of reliability has been adopted.

The next three terms, resiliency, vulnerability and robustness have been applied specifically to water supply systems.

2.2.5 Resiliency

The first of these three terms was introduced by Fiering [97] and concerns the concept of 'resiliency'. Resiliency concerns the ability of a water resource system to recover from critical situations. Fiering defined resiliency as :

> "A measure of a system's ability to avoid the boundary locus (failure of the system) and if forced over that edge (into a failure state), to sustain a controlled downward trajectory or to recover to its initial state."

Fiering suggested that the resilience of a water resource system is implicitly linked to time. He formulated several alternative measures that could be used to indicate the resilience of a system. These alternative measures included :

- Residence time in a nonfailure state.
- Expected economic outcome.
- Steady state probability of not being in a failure state.
- Mean passage time to a failure state.
- Mean passage time from non-failure to failure states.
- Mean passage time between failure states.
- Mean passage time to failure from complete recovery.

It is noted that the definition given by Fiering for resilience of 'the steady state probability of not being in a failure state' is the same as Hashimoto's definition for reliability.

Hashimoto et al. [130] defined the term resiliency as "how quickly a system is likely to recover or bounce back from a failure once a failure has occurred". If system failures occur over an extended period of time with only slow recovery this will have a serious impact upon the users of the system. Moy et al. [214] defined resilience as "the probability of recovery from failure to some acceptable state within a specified time interval". Simonovic [275] used resiliency to describe the ability of a reservoir to return to a satisfactory state given that a failure has occurred. His definition included both the failure occurrence and duration.

Within this thesis, Hashimoto's definition of resiliency has been adopted.

2.2.6 Vulnerability

Vulnerability is a measure of the significance, extent or consequences of failure of a system.

With reference to a water supply system, Hashimoto et al. [130] defined the term vulnerability as the "likely magnitude of a failure if a failure occurs". He noted that efforts aimed at maximising a water supply system's efficiency and reliability could also increase the system's vulnerability to costly failure, should failure occur. Weeraratne et al. [315] defined vulnerability as "a measure of the severity of failure in a system", and noted that there was a trade-off between system reliability and vulnerability. Simonovic's [275] definition of the term vulnerability related to the maximum penalty resulting from system failure over a given period of time.

It is important that decision-makers be aware of the vulnerability of a system to severe failure should a failure occur.

Within this thesis, Hashimoto's definition of vulnerability has been adopted.

2.2.7 Robustness

In engineering, project designs and operating policies that are sufficiently flexible to permit their adaption to a wide range of possible future conditions at little additional cost are referred to as 'robust'.

Fiering [95] first applied the term 'robustness' to the field of water resources planning. He used the term to describe whether or not the optimal project design parameter values would remain unchanged if future demand conditions were to vary from those initially assumed in the project design.

Hashimoto et al. [130] adapted this term to consider the sensitivity of total system cost. Hashimoto et al. used robustness to describe the overall economic performance of a water resources project.

The robustness of water supply systems has not been considered in the work contained within this thesis. It is however an important term in reliability aspects of the design of water supply systems and has been described here for completeness.

2.3 Risk Analysis

There are many definitions for the term "risk analysis" but in general it addresses the probability and consequences of possible events.

Risk analysis has been widely applied and has evolved into a large field involving complex mathematical and statistical methods to quantitatively determine levels of risk. Risk analysis has been used in a large number of areas including agricultural production, drug research, waste disposal and management, water management, public and environmental health, computer security, energy systems, transportation systems, climate change, natural hazards and technological hazards. In each of these areas, risk analysis has been used as a tool to evaluate risks and determine strategies for their management. Risk analysis requires reasoning about the potential occurrence of undesirable possible future events and seeks ways to avoid or delay them.

The following tale taken from a paper on risk management, illustrates some of the problems associated with risk assessment and analysis.

The Lady or the Tiger

> The young man could open either door he pleased. If he opened the one, there came out of it a hungry tiger, the fiercest and most cruel that could ever be procured, which would immediately tear him to pieces. But if he opened the other door there came forth a lady; the most suitable to his years and station that His Majesty could select among his fair citizens. So I leave it to you, which door to open ?
>
> The first man refused to take the chance. He lived safe and died chaste.
>
> The second man hired risk assessment consultants. He collected all the available data on lady and tiger populations. He brought in sophisticated technology to listen for growling and detect the faintest whiff of perfume. He completed check lists. He developed a utility function and assessed his risk averseness. Finally sensing that in a few more years he would be in no condition to enjoy the lady anyway, he opened the optimal door. And was eaten by a low probability tiger.
>
> The third man took a course in tiger taming. He opened a door at random and was eaten by the lady.

Originally obtained from Stockton [284] and quoted by Clark [38].

The risk analysis process can be divided into three components :

- Risk Identification

  Risk identification involves the recognition of the potential hazards that exist and some definition of the characteristics of these hazards. Kaplan and Garrick [163] describe risk identification as 'the process by which a hazard is recognized and the attempt is made to define some of its physical characteristics'.

- Risk Estimation

  Risk estimation involves some quantification of these hazards and an understanding of their causes and effects.

- Risk Evaluation

  Risk evaluation involves judgement on the significance and acceptability of the identified risks, their probability, and evaluation.

Various techniques have been developed in the wide range of fields in which risk analysis has been applied. Each of these techniques comprise some or all of the three components of the risk analysis process. The following sections provide a review of some of the more widely known techniques that have been used in risk analysis.

2.3.1 Preliminary Hazard Analysis

In the preliminary design stage of a project for which risk analysis is to be a part of the overall project design and operation, a preliminary hazard analysis is often undertaken.

Preliminary hazard analysis was first used in the United States in the context of safety analysis of missiles powered by liquid fuels as detailed by Hanmer [124]. The technique was later formalised in the aeronautical industry (Henley and Kumamoto [133]), and has since been used in the chemical (Powers and Tompkins [237]), nuclear (Roberts et al. [252], Reny et al. [247]), and aerospace (Henley and Kumamoto [133]) industries.

A preliminary hazard analysis involves the preliminary identification of the system elements or events that lead to hazards. The technique includes a qualitative consideration of the event sequences that transform a hazard into an accident, together with corrective measures and consequences of the accident.

The consequences of the accident are often classified according to their severity, for example, within the aerospace industry, the hazard severity has been divided into four categories (Henley and Kumamoto [133]) :

- Category 1 - negligible
- Category 2 - marginal
- Category 3 - critical
- Category 4 - catastrophic

Having identified various possible hazards, accident prevention measures are considered to eliminate category 4 and possibly category 3 and 2 hazards. These measures may include equipment design changes, redirection of goals or functions, or contingency actions (such as protection systems).

In general, preliminary hazard analysis is a technique used to identify the major system components and events which can lead to hazards, while the system is still in a preliminary design stage.

2.3.2 Statistical Analysis of Historical Events

When a risk-causing event or series of events has occurred historically and is clearly identifiable, then statistical analysis of these events can be undertaken. Providing these past events have produced consequences at least as large as those of concern in the risk assessment being undertaken, then the result of the statistical analysis can be directly applied. Statistical analysis has been used to estimate various accident and natural event statistics and these data have been recorded in a range of reference documents (for example, National Safety Council [222] and Cohen and Lee [44]).

The frequency of a risk-causing event is estimated by taking the number of event occurrences in some time period and dividing by the appropriate 'exposure time' in that period as given in Equation 2.1.

$$F = N/D \tag{2.1}$$

where,

F	=	The estimated event frequency.
N	=	The number of events which have occurred in some past time period.
D	=	The total 'exposure time' in the observed time period.

The frequency obtained from Equation 2.1 should be treated as an average. If the frequency is assumed constant over time then the expected number of events is simply the frequency multiplied by the time period. It must be recognised, however, that in various situations, failure frequencies will vary over time (for example, the failure frequency of a motor towards the end of its design life).

If events have varying consequences, a plot of frequency versus consequence can be constructed. A curve of this form is called the 'complementary cumulative distribution function' (CDDF). From this function the frequency-averaged consequence $\overline{C}$ can be computed as given in Equation 2.2.

$$\overline{C} = \sum_{i=1}^{n} F_i C_i \tag{2.2}$$

where,

$\overline{C}$	=	The frequency-averaged consequence.
F_i	=	The frequency for events having consequences in a certain interval (i).
C_i	=	The characteristic consequence of the interval (i).

Uncertainties associated with frequency-consequence curves can be calculated using standard statistical techniques.

2.3.3 Extrapolation Techniques

When undertaking a risk assessment, it is often required to consider an event with a specified recurrence interval. For example, it may be necessary to design a bridge to pass the one in two hundred year flood. When the record of observed events does not contain an event of this magnitude, extrapolation of the available data may be necessary. Extrapolation techniques are used to estimate the frequency of severe events by smoothly extrapolating from less severe events which have occurred in the past. When applying extrapolation techniques, the more severe events are assumed to be caused by the same mechanisms and processes as the less severe events. A continuous behaviour in the processes and consequences from less severe to more severe is assumed.

Extrapolation techniques are used to determine the frequencies of extreme floods from a streamflow record containing only minor floods, catastrophic earthquakes from minor earthquakes and extremely high winds from occurrences of less severity.

It has been shown that, if the consequence of the risk-causing event is related to the maximum or minimum value of some underlying process, and certain assumptions are valid, then a theory called 'extreme value theory' can be applied to extrapolate the event frequencies (Gumbel, [118] and Galambos, [101]). By viewing catastrophic events as being the maximum of a sequence of recorded happenings, extreme value theory has been applied to a variety of problems. For example, the prediction of flood height exceedance probabilities, maximum fire consequences, and maximum hurricane consequences.

The basic estimation process involved in all these extrapolation techniques is to describe the frequency versus consequence by a function and extend the application range of the function to predict the frequencies of events with extremely large consequences which have not yet been observed. When extrapolating beyond the largest observed consequence, the key factors in the accuracy of these extrapolations are (1) the validity of the function and (2) the assumption of smooth continuous behaviour beyond the largest observed value.

It is important to calculate error bounds for the frequency versus consequence curve obtained which describe the uncertainties due to both the data and its extrapolation.

2.3.4 Event Tree Techniques

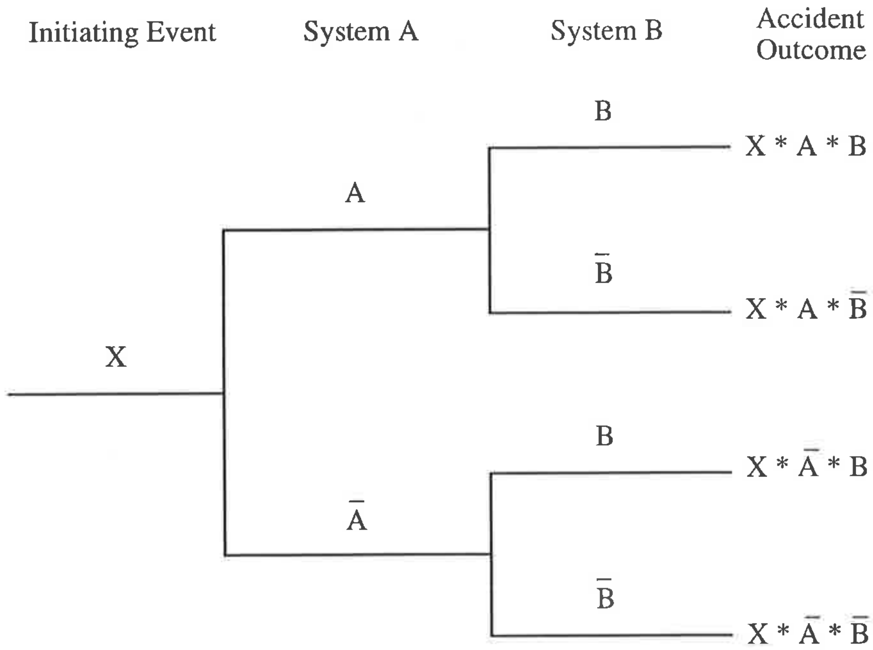
Event tree techniques, as described by Reid [245], are used to represent possible accident sequences involving the success or failure of individual system components. These techniques are an inductive logic approach which can be applied when a chain of events precede the occurrence of an accident (Vesely [305]).

An accident sequence consists of a defined event which triggers the sequence, followed by a chain of specific system failures. Associated with each accident sequence will be a specific accident severity. To identify all possible accident sequences, an event tree table is constructed of the form shown in Figure 2.1. A particular accident sequence in the event tree is then a particular path from initiating event through each system's performance state to the final accident outcome.

In practice, event tree techniques have been applied to analyse accidents at nuclear power plants, chemical plants, offshore oil facilities, and other industrial and military facilities. When applying event tree techniques to real problems, the event tree can become very complex comprising hundreds of thousands of accident sequences. In order to construct the event tree, the actual plant or facility must be studied to identify the different kind of accident-initiating events which can occur and the different systems which will be called upon to mitigate the accident.

Since event trees can be extremely large, it is helpful, wherever possible, to reduce the number of accident sequences which must be evaluated. Various reduction techniques exist which eliminate system forks which do not influence consequences or which cannot occur. Large event trees can also be subdivided into products of smaller event trees which can be analysed more simply.

Once an event tree has been constructed and reduced where possible, it can



where

A	=	Success of System A
$\overline{\text{A}}$	=	Failure of System A
B	=	Success of System B
$\overline{\text{B}}$	=	Failure of System B

Figure 2.1: Sample Event Tree

be used to quantify the probabilities of individual accident sequences. For example, from Figure 2.1 the probability of sequence $X * \overline{A} * B$ occurring is given by Equation 2.3.

$$P(X * \overline{A} * B) = P(X \text{ occurs}) * P(A \text{ fails}) * P(B \text{ succeeds given } A \text{ fails}) \quad (2.3)$$

where,

$P(I)$ is the probability that the event I occurs.

Typically, frequencies of the initiating events are determined using the statistical analysis techniques discussed in Section 2.3.2. Traditional reliability techniques are used to estimate system failures and successes when data exists. When data is not available, alternative techniques may be necessary such as fault tree analysis.

2.3.5 Fault Tree Techniques

A common approach used within the broad area of systems safety analysis is fault tree analysis (Roberts et al. [252], Vesely [305], Thomson [292], Reid [245], Misra [207] and many others). This technique is used to examine each possible outcome and to illustrate the various ways in which this outcome can be produced. Clearly the reliability of the fault tree will depend on the ability of those preparing the fault tree to perceive all possible happenings in the system that may contribute to the failure of the system.

Assuming the assessed fault tree covers all possible combinations of individual failures, then an estimate of the combined probabilities of the individual events can be made to produce a probability value for system failure.

Fault tree analysis is one of the principal methods used in the area of systems safety analysis. It can be used to identify the most likely causes of system failure in the event of a system breakdown. The first step involved in fault tree analysis is to construct a 'fault tree'. A 'fault tree' is a model that graphically and logically describes the various combinations of possible events, both fault and normal, occurring in a system that would lead to the undesired condition of the system under consideration called the 'top event'.

Before the fault tree can be constructed, the analyst must acquire a thorough understanding of the system and carefully define the 'top event'. In the construction of the fault tree, the sequences of events that lead to the undesired event are shown below the 'top event' and are logically related to the undesired event by AND and OR logic gates. The input events to each logic gate are also outputs from other logic gates. These events are developed further until the sequence of events leads to basic causes of interest called 'basic events'. These 'basic events' form the bottom tier to the fault tree and represent the limit of resolution of the fault tree. A sample fault tree is shown in Figure 2.2.

In this figure, basic events A and B are connected by an AND gate. For these two basic events to impact upon the system both must occur at the same time. Basic events C and D are connected by an OR gate. The occurrence of either of these two basic events will impact upon the system. For the top event to be reached, basic events A and B must occur at the same time as either basic event C or basic event D.

In order to develop fault flows in a fault tree, a structuring process is used as outlined by Haasl [119]. Three failure mechanisms are identified in the structuring process that contribute to a component being in a fault state. These mechanisms are :

1. A <u>primary</u> failure is a failure due to the internal characteristics of the system element under consideration.

2. A <u>secondary</u> failure is a failure due to excessive environmental or operational stress placed upon the system element.

3. A <u>command</u> fault is an inadvertent operation or nonoperation of a system

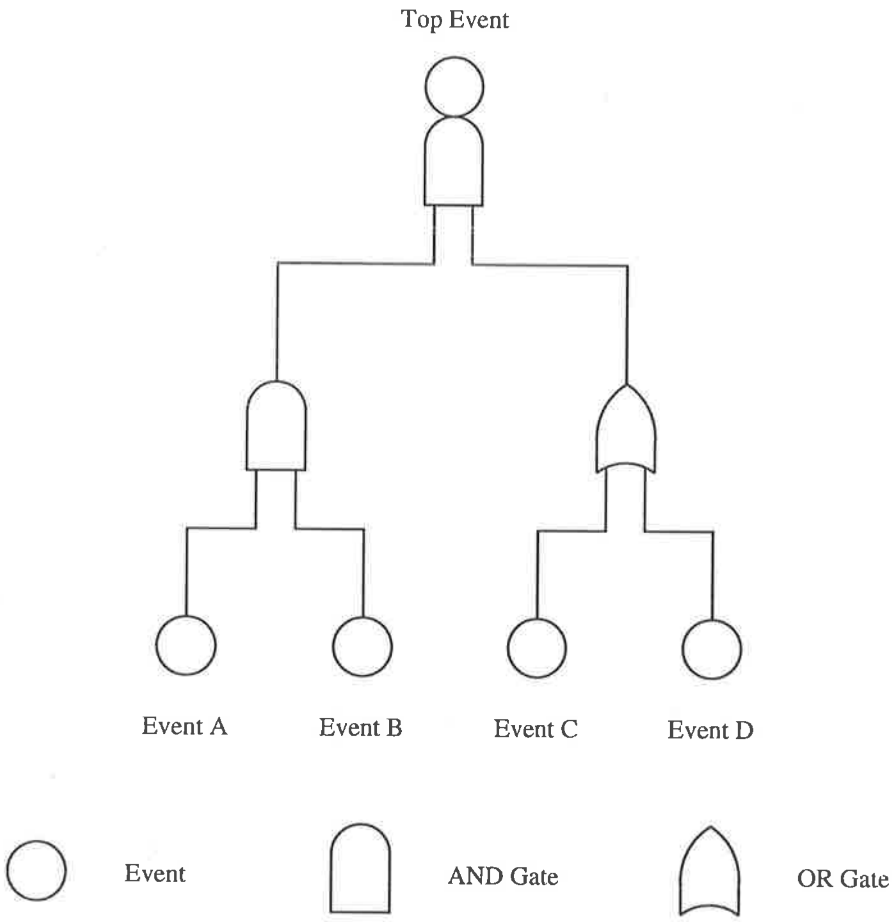


Figure 2.2: Sample Fault Tree

element due to failure(s) of initiating element(s) to respond as intended to system conditions.

Work has been carried out to automate the construction of fault trees, with Fussell [100] describing the automation of fault tree construction for an electrical system and Powers et al. [238] automating fault tree construction for a chemical system.

The fault tree, once constructed, can be used to determine possible causes of an accident and even to discover failure combinations which might otherwise have remained undiscovered. The fault tree provides a convenient and efficient format for evaluation both quantitatively and qualitatively of the probability of the occurrence of the 'top event'.

The technique of fault tree analysis has rarely been applied in the area of water supply systems. An application of fault tree analysis to the study of water shortages within a water supply system is given by Ikebuchi [154].

2.3.6 Probabilistic Risk Analysis

Probabilistic Risk Analysis (PRA) is a technique for assessing the potential for failure of complex, hazardous systems and identifying ways to reduce failure risks. The first major application of PRA was in the Reactor Safety Study, WASH-1400 [301] undertaken by the U.S. Nuclear Regulatory Commission. Since that time PRA has been used extensively in the nuclear generating industry and the chemical processing industry and has found application in many other fields. PRA is used to highlight potential technical malfunctions and human errors that can lead to a system failure. Examples of the application of PRA can be found in many papers for example Reny et al. [247], Vick and Bromwell [307], Stewart [283] and many others.

Within the field of water engineering, PRA has been used in the area of dam safety to identify the principal scenarios which may cause a dam to fail, and the combination of events which must happen for the failure to occur. In dam safety evaluation, Nielsen et al. [221] has suggested that PRA can be used to assess the following factors :

- Extreme floods and earthquakes.
- Physical resistance of dams and probability of failure.

- Nature of dam failure, if the loading conditions exceed the resistance of the dam.

- Breaching characteristics of dam failure and the resulting consequences.

The level of understanding of all possible events that can lead to dam failure, and the consequences of such failure will always be limited in extent. PRA can however be used to greatly improve the available knowledge by providing insights into the areas of weakness and vulnerability for the dam. Fault tree analysis can be used in conjunction with PRA to assist in quantifying a dam's level of vulnerability.

Nielsen et al. [221] detailed the inclusion of PRA within the dam safety program at British Columbia Hydro. The dam safety program includes periodic inspection and review of all dams in the system and where necessary, investigations are undertaken into suspected deficiencies. PRA techniques have been integrated into the dam safety review process framework as shown in Figure 2.3 (from Nielsen et al. [221]) through consequence-based safety evaluations for floods and earthquakes.

In each scenario, the proposed dam safety criteria was made proportional to the consequences of failure downstream of the dam. PRA emphasises the relative importance of each potential failure mode, whether or not it is possible to calculate the factor of safety for the mode. This ensures that important issues are not overlooked because they may be difficult to address quantitatively. PRA could be described as failure-driven rather than analysis-driven and is based upon the conditions, peculiarities and associated failure modes unique to each dam, addressing each situation with an appropriate analysis technique.

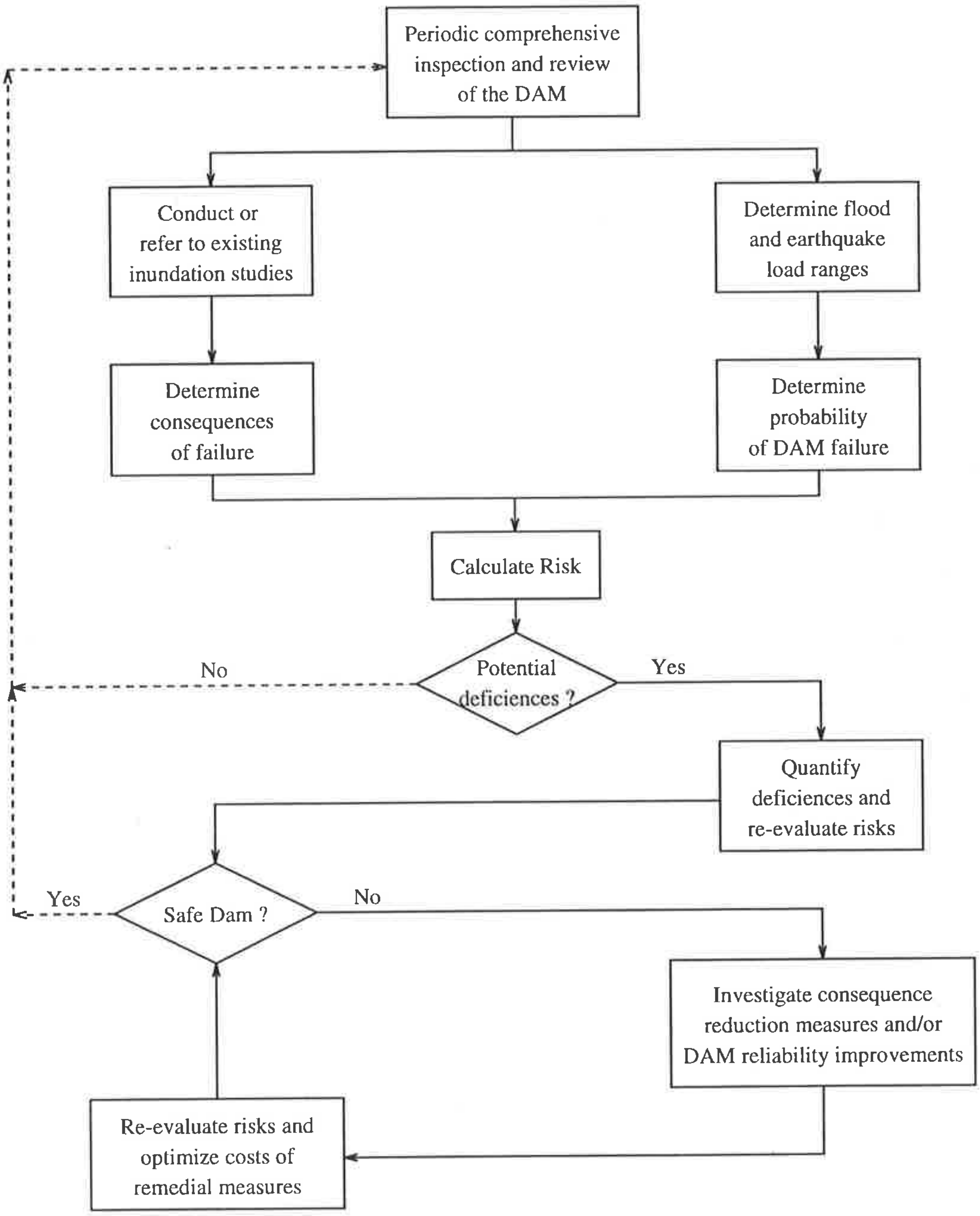


Figure 2.3: Consequence-based Dam Safety Evaluations and Improvements (Nielsen et al. [221])

A methodology has been outlined by Nielsen et al. [221] for the risk analysis of a particular dam. The methodology is based around a workshop format over a period of four to five days, and draws together experts having in-depth knowledge of one or more aspects of design, analysis or performance of the dam under study. Discussion throughout the workshop is compiled and becomes part of the permanent record regarding the dam.

The following steps make up the methodology.

1. Review the field performance experience.
2. Inspect the dam site.
3. Identify the potential dam failure modes.
4. Construct an 'event tree' for potential dam failure modes.
5. Assess the probabilities associated with each of the 'event tree' components.
6. Interpret the results obtained.
7. Iterate the process.

PRA is a useful tool in risk analysis. By addressing a comprehensive range of extreme events, safety judgements can be expressed as probabilities of failure. PRA can provide insights into the impact of extreme events by focusing on the hazards and mechanisms that produce them. Within the framework of the assessment of water supply system reliability, aspects of PRA may be useful. The application of PRA in dam safety analysis focuses on extreme low probability events such as major earthquakes or floods approaching the Probable Maximum Flood (PMF). When considering water supply system reliability, such extreme events are likely to cause widespread disruption to the community and are considered beyond the scope of the current research.

2.3.7 Frequency Duration Analysis

Frequency duration methods were first developed within the field of power system engineering to evaluate and compute system reliability for electric power generation, transmission and distribution. Hall et al. [126] described the application of this technique to power generating machines in parallel. They compared the results obtained using frequency duration analysis with methods used by Halperin and Adler [127] and a technique discussed by Sauter et al. [267] for 20 identical machines and 22 machines of varying capacities. Following the paper by Hall et al., Ringlee and Wood [251] applied the technique in a power demand model and a capacity reserve model.

This technique has been adapted from the power industry and applied to the analytical simulation of a water supply system in two papers by Hobbs and Beim [14] [137]. In the first of these papers, three versions of the frequency-duration approach were presented to determine the unavailability and expected unserved demand of a water supply system. In the companion paper, a Markov chain approach was also considered and this approach together with the frequency-duration approach compared with the more realistic Monte Carlo simulation approach. This comparison verified that the analytical techniques produced useful reliability index estimates.

The major difficulty encountered in the application of the frequency duration technique to water supply systems is the effect of significant storage capacity cannot easily be included in the reliability calculations. Further details of this technique and its limitations are described in Section 2.4.4.4 of this chapter and Section 3.5 of Chapter 3.

2.3.8 Failure Mode and Effects Analysis (FMEA)

Failure mode and effects analysis (FMEA) is an inductive analysis that examines, on a component by component basis, all possible failure modes and identifies the resulting effect of these failures on the system. FMEA is a more detailed form of fault tree analysis since every mode of failure of every component must be considered. FMEA was first applied in the field of aircraft safety (Recht, [244]), and has since been applied to a diverse range of fields including, the aerospace (Krishan and Pauperas, [170], Reinhardt, [246]) , nuclear (United States Nuclear Regulatory Commission, [302]), chemical (Lees, [181] [182]), and car manufacturing (Yamada, [323]) industries.

Villemeur [308] describes the four main steps in FMEA as :

1. Definition of the system, its functions and components.
2. Identification of the component failure modes and their causes.
3. Study of the failure mode effects.
4. Conclusions and recommendations.

Villemeur presented these four steps in the form of a flow chart as shown in Figure 2.4.

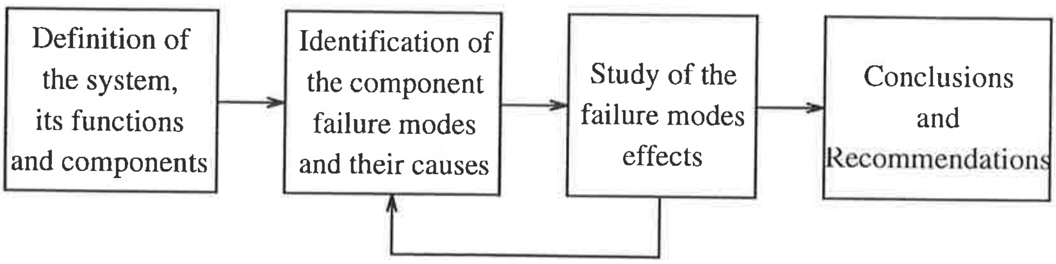


Figure 2.4: Flow Chart for a Failure Mode and Effects Analysis

The use FMEA is required by some standards, for example IEEE 352-1975 [155] in relation to the nuclear power industry.

A natural extension of FMEA is the failure modes, effects and criticality analysis (FMECA) as described in detail by Jordan and Marshall [160]. This technique assigns probabilities to each of the failure modes and analyses the severity level of the effects of failure. The critical failure modes are those having the highest combined failure probability and severity levels.

2.3.9 Hazard and Operability Analysis (HAZOP)

Hazard and operability analysis (HAZOP) can be described as an extended form of the FMEA technique that incorporates aspects of cause-consequence analysis (Villemeur, [308]). Thomson [292] describes HAZOP as an 'inductive' technique in contrast to fault or event trees, which he terms 'deductive' techniques. The extensions to the FMEA technique include a range of operability factors in addition to the equipment fault modes. HAZOP was initially developed by Imperial Chemical Industries (Lawley, [178] [179]) and now finds its primary use in the chemical industry for the examination of the operability of chemical processes. Using the complete details of a proposed chemical plant, each part of the process is examined for design intention, potential variations from this intention, causes for such variations, and consequences of such variations.

Each part of the plant is assessed using a table of 'guide words'. These guide words include :

NO, NOT or NONE	:	The design intention is not achieved (eg. no flow, reversed flow, no electric current)
PART OF	:	Only some of the design intentions are

		achieved.
AS WELL AS	:	An additional activity occurs together with the design intention.
REVERSE	:	The opposite of the design intention occurs.
OTHER or OTHER THAN	:	Some completely different outcome to the design intention occurs.
MORE	:	Increased or excessive temperature, pressure, flow rate, viscosity etc.
LESS	:	Insufficient or reduced temperature, pressure, flow rate, viscosity etc.

As each part of the plant is assessed, all possible failure causes and their effects are listed with the help of the guide words. If a failure is seen as important because of the probability of its occurrence, or the consequences of its effects, the appropriate measures necessary to lower its probability or reduce its effects are specified.

The technique has the advantage that those components whose failure effects are readily reduced can be quickly detected. Unlike FMEA, HAZOP does not require the systematic study of the failure modes of each component and of their effects.

2.3.10 Cause-Consequence Analysis

Cause-consequence diagrams were invented at the RISØ Laboratories in Denmark (Nielsen [220]). The construction of these diagrams commences with a choice of critical event. Critical events are chosen so as to be convenient starting points for analysis, with most sequential problems following the critical event.

Having selected the critical event, a 'consequence tracing' process is undertaken following each of the possible chain of events through the system. As the chain of events are traced, a branch taking two paths may be encountered. Each of these branches is followed until a consequence is reached. Each chain of events may take alternative forms, depending on different conditions.

The construction of a cause-consequence diagram is carried out by taking each event (commencing with the initial event) and asking :

- What conditions will cause this event to lead to further events ?
- What other conditions would produce different events ?
- Which components does this event impact upon ?
- What further events does this event cause ?

As additional events are determined the same set of questions are asked of each new event.

The cause-consequence analysis technique is a combination of fault tree analysis (used to show causes) and event tree analysis (used to show consequences).

2.3.11 Failure Tolerance Analysis

Failure tolerance analysis has been used in risk management for the space station freedom program as described by Krishan and Pauperas [170]. Using this technique, the criteria for system survival, crew safety and mission success have been established. Failure tolerance analysis identifies all modes of failure within a system design and is the first step towards identifying potential critical failures.

The technique involves the systematic evaluation of item failures and the resulting consequence. Each failure is then ranked according to the severity of the failure consequence.

2.3.12 Summary

In this section, some of the techniques that have been used to undertake risk analysis in a broad range of fields have been reviewed. In each of these fields, risk analysis has been used as a tool to evaluate risks and determine strategies for the management of these risks. The review given in this section does not seek to cover the complete range of techniques available in risk analysis. Rather, the review gives a description of some of the more widely known techniques.

2.4 Application of Risk Assessment to the Operation and Management of Water Supply Systems

Planners and operators of water supply systems in the past, have rarely given specific consideration to tradeoffs between reliability and cost (Cullinane [53]). Rather, "they have viewed their responsibility as efficiently meeting water demands when and where they occur in their systems with the highest level of reliability achievable" (Hobbs [138]).

In recent years, the techniques associated with risk analysis have steadily evolved. Although research has been carried out examining the economic tradeoffs between reliability and cost (Damelin et al. [57]), the application of this research to the operation and management of water supply systems has been limited.

It is important to remember that the application of risk analysis to any water supply system will not of itself reduce or eliminate risk of failure for the system. Within certain confidence limits the technique merely provides an indication of the reliability-cost tradeoffs for the system.

Planners now recognise that large expansions of their water supply headworks systems are difficult due to limited potential sites and public sensitivity to the impact of such projects. Rather, they must endeavour to manage their systems more efficiently and effectively. The consideration of trade-offs between reliability and cost is becoming increasingly important as highlighted by Boland et al. [21], Lauria [176] and Prasifka [239]. Looking to the future, this trend is expected to increase.

Hobbs [134] argues that, in order to provide accurate assessment of these trade-offs, recognition must be made, not only of the variability of reservoir inflows, but also of other important sources of unreliability such as pipeline, pump and equipment failures, water supply contamination and demand uncertainty.

When examining the reliability-cost tradeoffs for a water supply system, three elements form part of the examination. The first element is the model used to represent the system. The second element is the estimation of parameters that are used within the system model. The third element are the indices that are used to assess the reliability of the modelled system. In the following sections, each of these three elements are described in detail. With each description, a review of related research undertaken in the area is given.

2.4.1 The System Model

When modelling a water supply system, it is helpful to divide the system into five components as shown schematically in Figure 2.5.

These five components directly affect the reliability of the system and are described in further detail in the following sections.

2.4.1.1 The Water Source

For all water supply systems a source of water is necessary. This source may be a river or water body, a local catchment area or a ground water basin. The reliability of this water source both in quantity and quality will have an impact on the overall reliability of the system. Algal outbreaks in the river or water body, droughts in the local catchment or contamination of the ground water source will all affect the reliability of the system.

Datta and Houck [70] present an example of how the reliability of a water source can be assessed. They develop a stochastic optimisation model for the real-time operation of reservoirs and consider reliability measures on the basis of errors in inflow forecasting.

2.4.1.2 The Bulk Water Transmission, Pumpage and Treatment

These elements of a water supply system have been considered together because failure of one or more of these components will result in a 'capacity' rather than 'storage' failure of the system.

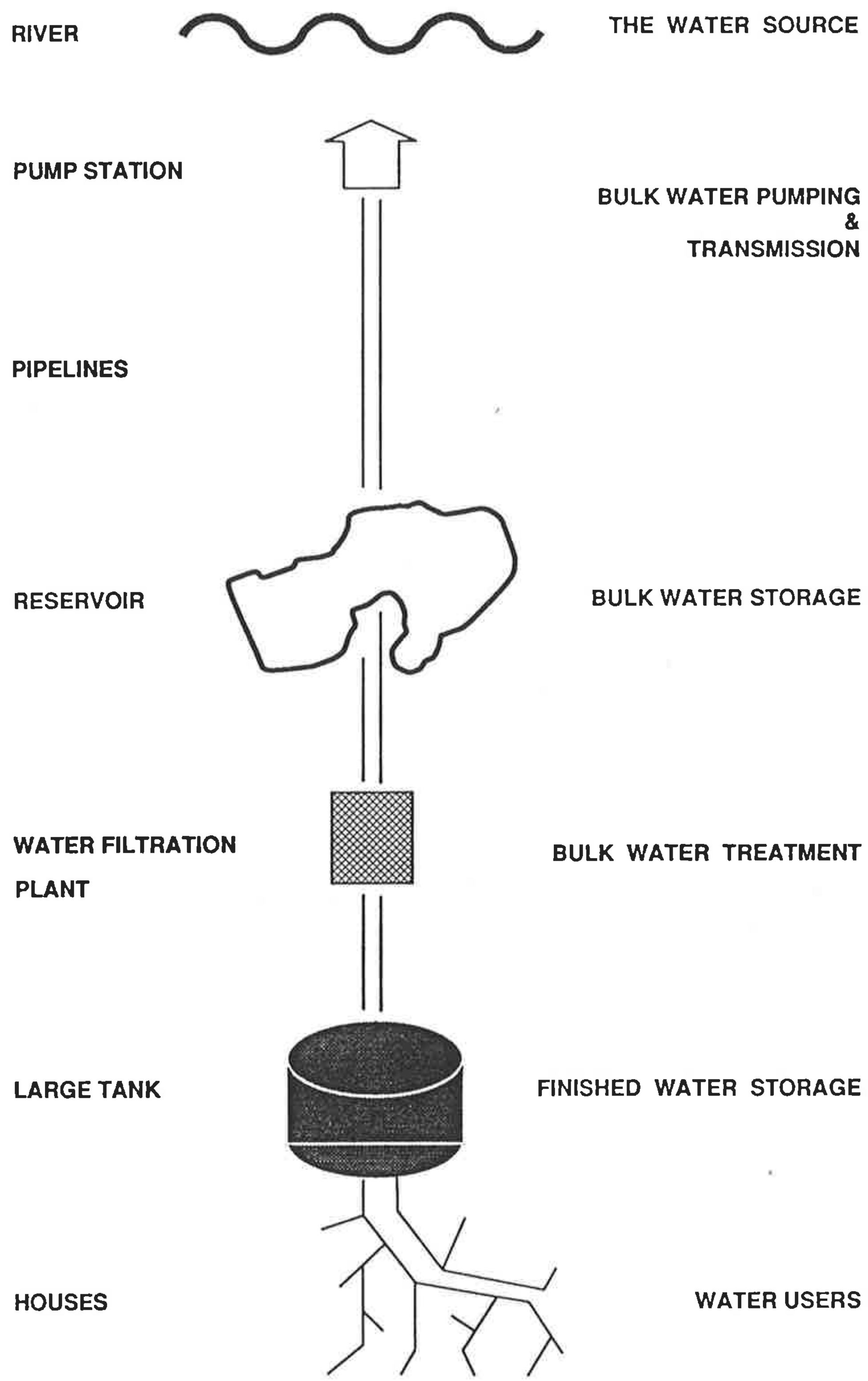


Figure 2.5: An Idealised Headworks System

Evaluation of the reliability of these components can be classified as either analytical or Monte Carlo simulation. Analytical techniques represent the system by a mathematical model and evaluate the reliability indices from this model using mathematical solutions. Monte Carlo simulation methods, however, estimate the reliability indices by simulating the actual processes and random behaviour of the system. The method therefore treats the problem as a series of real experiments. There are merits and shortcomings in both methods. Generally Monte Carlo simulation requires a large amount of computing time, however it can include any system effect or system process which may have to be approximated in analytical methods.

Some research has been undertaken in this area. Damelin et al. [57] used Monte Carlo simulation to examine the reliability of a water supply system supplying a deterministic demand pattern in which shortfalls were caused by random failure of the pumping equipment. Streamflows were not considered by Damelin et al.

Wagner et al. [310] examined a distribution network in which pipes and pumps were subject to random failures. These failures were described by probability distribution functions with user specified parameters. Using Monte Carlo simulation, random failure and repair events were generated and the system simulated using a hydraulic network model. A variety of reliability measures were calculated from the simulation of the system relating to the number, location, duration and effects of failures. Wagner reduced the simulation time for the model by applying a variance reduction technique.

Shamir and Howard [269] presented an analytical technique to determine the reliability of a dual source reservoir/aquifer water supply system. Economic loss caused by demand shortfalls were assessed when considering capacity expansion to improve supply reliability. They noted that when the system became more complicated, it was no longer possible to compute the probability distributions. Analytical expressions for the reliability measures could only be obtained by making certain assumptions which reduced the complexity of the problem.

Cullinane [53] considered pumping stations and water mains forming part of a water distribution system. He presented an analytical reliability evaluation approach applicable to these components. Cullinane used time-to-failure and time-to-repair data to develop simplified techniques for the evaluation of component reliability.

Equipment failures are typically assumed to occur randomly and to be conditionally independent. Conditional independence means that the probability of a specific piece of equipment being available at a given point in time is independent of the condition of the other components and the level of demand. It is possible to consider a degree of dependence in some of the approaches. For example, if a 'power supply' component is considered in series with a set of pumps.

2.4.1.3 The Bulk Water Storage.

When water is obtained as runoff from a local catchment and that runoff has significant monthly variation, there will be a need for some form of bulk water storage in the form of a dam or reservoir. The probability of failure of an existing storage for a given time period, is dependent on the volume in storage and the uncertainty in future streamflows and demands.

2.4.1.4 The Finished Water Storage.

The purpose of finished water storage capacity is to provide balancing storage to meet peak demands such as fire fighting requirements or peak day demands.

Very little research has been carried out examining the tradeoffs between reliability and cost associated with the design and operation of these storage facilities. Because the costs associated with maintaining minimum storage levels for fire protection may be considerable for large cities, there may be potential for significant cost savings.

Hobbs [139] suggested that the reliability of both the bulk and finished water storages can be determined using a variety of techniques. These include :

- Monte Carlo simulation of inflows assuming a predetermined set of demands and operating rules and determining the average reliability for the system (eg., Loucks et al. [190]).

- Assuming inflows are represented by a Markov process and using a predetermined set of demands and operating rules. The resulting model is then solved for the steady state probabilities for varying storage levels (eg., Loucks et al. [190], Beim and Hobbs [14]).

- Using dynamic programming or some other form of mathematical programming to optimise the operation of the storage, based on an assumed inflow probability distribution function (eg., Loucks et al. [190], Palmer et al. [227]).

- Assuming the reservoir commences a failure event at a specific storage level and applying a modified frequency duration analysis approach (eg., Hobbs et al. [136]).

2.4.1.5 The Water Users.

Future water demands are variable and are a function of climatic conditions, consumer preferences, income levels and the level of economic development.

These factors are uncertain and in any reliability assessment they should in some way be considered.

In order to forecast future demands, time series analysis has been applied by many researchers to municipal water use (eg. Wong [320], Young [330], Willsie and Pratt [318], Yamauchi and Huang [324], Maidment and Parzen [193]). The paper by Maidment and Parzen [193] describes the development of a time series model of monthly municipal water use which considers four factors affecting water use : trend, seasonality, autocorrelation and climatic correlation. Researchers typically consider these demand uncertainties using sensitivity analysis.

Researchers have often included the effect of climatic conditions by separating the year into two seasons. The winter season consisting of mostly indoor water use while the summer season including both indoor and outdoor water use (eg. Howe and Linaweaver [148], Carver and Boland [29], Boland [21]).

It is generally considered that the time structure of water use is best modelled using time series analysis.

To include the impact of demand uncertainty in a water supply reliability framework, Hobbs [139] proposed the application of a 'demand duration curve' adopted from reliability studies in the power industry, as used by Billinton and Allan [18]. Frequency of demands at different levels could be determined from hourly demand records and used to calculate the frequency and duration of outages using reliability methods. Uncertainty in future population forecasts and economic conditions could be included by assuming prior probability distributions for these parameters. Hobbs [139], however, made the observation that the only rigorous way to examine the impact of demand management measures was using a chronological simulation model that explicitly considers those effects.

When seeking to determine tradeoffs between reliability and cost for a water supply system it is important to assess the economic cost of shortages to consumers. This assessment is addressed in Section 2.4.3 under the topic of economic costs.

2.4.2 Parameter Uncertainty

When any analysis of a water supply system is undertaken, there will always be some uncertainty in the estimates of the various parameters used within the models representing the real situation. In some cases the order of magnitude of parameter uncertainties may be quite large (for example, in the estimation of the failure frequency for a component of the bulk water transfer system). It is important that some assessment of the relative impact of these uncertainties be included in the overall reliability of the system under consideration.

There are a number of approaches that can be adopted to deal with parameter uncertainties. These include :

- Ignoring these uncertainties.

  Where the uncertainty associated with a particular parameter has little impact on the overall reliability assessment it is valid to ignore these uncertainties. As the impact of the parameter uncertainties and/or the complexity of the model increases, ignoring the parameter uncertainty is no longer satisfactory.

- Adopting the 'worst-case' situation.

  Adoption of the 'worst-case' scenario has some merit, particularly when the results are dependent on one or two sensitive parameters, as it builds in a 'margin of safety'. When the results are dependent on a large number of independent sources of uncertainty, the final result will be overly pessimistic.

- Including the uncertainties within the models.

  When the reliability assessment models are relatively simple, it is possible to include the parameter uncertainties within the model. When the models become complex, the inclusion of parameter uncertainties becomes difficult as the models become unwieldy.

- Using the best estimates for the parameters and undertaking sensitivity analysis on the critical parameters.

  Although not as rigorous as including the uncertainties within the model, sensitivity analysis undertaken on the critical parameters can be used to highlight the impact of the parameter uncertainties.

We will now consider various approaches that have been used to include parameter uncertainty within models used for the reliability assessment of water supply systems. All of the approaches adopted within this area have employed numerical methods. Management of uncertainty in the field of artificial intelligence have also used non-numerical (symbolic) methods as described by Uckun et al. [299]. These may be applicable to water supply systems but to date have not been considered by researchers.

The use of synthetic inflow and demand sequences in reservoir system simulation has been used by many as a means of improving the ability to estimate the reliability of a water supply system. In order to generate these synthetic hydrologic sequences, statistical parameters need to be obtained representing the statistical properties of streamflows and demands. These parameters are then used to generate the sequences in conjunction with appropriate streamflow and demand data generation models. Typically these parameters have been estimated from historical streamflow and demand records. Inherent in these estimates will be uncertainties due to the limited length of record and data recording errors.

Simulation methodologies have been developed which incorporate the uncertainty of these parameters. McLeod and Hipel [202] first presented an approach for generating synthetic annual river flow data that incorporated the uncertainty of the model parameters. Their paper described the application of this technique to the design of reservoirs.

Stedinger and Taylor [280] extended the work of McLeod and Hipel to simulation procedures for generating monthly rather than annual streamflow data. Their paper illustrated the impact of the uncertainty of parameters describing the distribution of annual flows on capacity-reliability relationships using two different monthly synthetic streamflow generation models.

Ng and Kuczera [223] enhanced the approach to consider uncertainty in both streamflow and demand data generation models. They examined four aspects of demand comprising natural, climatic, socioeconomic, and model parameter uncertainty. Results from their work highlighted the need to consider both streamflow model parameter and demand uncertainties when examining the reliability for a water supply headworks system.

As the number of parameters directly affecting the model uncertainty increases, it becomes difficult to consider all possible combinations of these parameters. An alternative is to undertake sensitivity analysis on key parameters, and assess the impact of these individual parameter uncertainties on the overall system.

2.4.3 Reliability Indices

Having developed a model to represent the water supply system, it is necessary to measure the performance of the system under various operating conditions.

In the current context, the reliability performance of an existing system is to be considered.

There are a number of indices that have been employed to measure the reliability of a water supply system (Hobbs, [134] [135] [137]). These include :

- A specific set of contingencies

  If the water supply system can satisfy a specific set of contingencies, say for example the most extreme drought on record, the system may be deemed reliable. Nardini et al. [217] described work in which the system manager, by the selection of a set of inflow sequences together with a real-time operations model, could determine a trade-off between the two criteria of risk-aversion (to avoid dramatic failures) and average-performance optimisation (to obtain the best long-term average performance). The disadvantage with reliability indices of this general form is that the selection of the contingency to be satisfied is rather arbitrary and different systems with differing probabilities of failure may not be discernible using these measures.

- A specified low probability of failure

  If the probability that the system will fail is less than a specified value, then the system is deemed to be reliable. This form of index has been widely used as described by Loucks et al. [190]. Hashimoto et al. [130] used an index of this form to define reliability. Weeraratne et al. [315] used Hashimoto's definition to examine the Grand River system. Palmer and Lettenmaier [229] used screening models to select critical streamflow sequences from synthetic streamflow data and then applied an optimisation model to generate cumulative distribution functions of system reliability.

- Severity Indiccs

If the system is able to satisfy a minimum ratio of available supply to demand, then the system is deemed to be reliable. These indices in effect measure how large failures are. Shamir and Howard [268] defined water supply system reliability in terms of shortfalls relative to the desired demand. This overall reliability was considered to depend on two components, total volume shortfalls and supply rate shortfalls.

- Frequency and Duration Indices

If the system is able to satisfy a lower limit of frequency and duration of restrictions then the system is deemed to be reliable. Shamir and Howard [268] described an analytical approach to the frequency and duration analysis of water supply systems. Hashimoto et al. [130] used an index of this form in their definition of resiliency. Hobbs [135] applied frequency and duration reliability methods developed by the electric power industry to estimate bulk water system capacity reliability. Hobbs and Beim [137] in their first of two papers presented three analytical simulation models for estimating the reliability of bulk water supply systems based on the frequency-duration analysis technique.

- Economic Costs

The economic costs associated with supply shortages depend on a number of factors and are often difficult to quantify. Limited research has been undertaken in this area. Dandy [61] [66] proposed a theoretical expression for economic cost of time restrictions on water use consisting of a loss of consumer surplus and a loss of producer surplus. The expression does not consider external costs of restrictions such as 'browning' of a city.

Using the estimates of economic costs to consumers, a system is deemed reliable if the economic consequences of the frequency and duration of failures of the system are less than a specified value. Hashimoto et al. [130] used an index of this form in their definition of vulnerability. Weeraratne et al. [315] considered yearly opportunity cost and compared economic considerations with a probability and frequency index. Loaiciga and Mariño [185] presented a reservoir planning model that produced a set of alternative feasible release policies, together with the corresponding trade-off curve between expected revenue against the standard deviation of such revenues.

Historically many of the first reliability studies carried out on water supply systems involved some form of contingency analysis. The shortcomings of this approach relate to its consistency between water supply authorities as the length of record and the events contained within the record may mean the 'event' considered can yield markedly different probabilities of failure between different water supply systems. For this reason, planners within water supply authorities have moved away from this approach towards probability based reliability indices. These indices still have some shortcomings in that the selected reliability level is often rather arbitrary. A more recent development has been to optimise the level of reliability by balancing the economic effects of supply shortfalls against the costs of operating a system in such a manner as to alleviate these shortfalls.

Moy et al. [214] explored the tradeoffs between sets of proposed reliability criteria using these criteria as components of an objective function within a multiobjective programming model. Burn et al. [24] examined the risk-based performance criteria introduced by Hashimoto of reliability, resiliency and vulnerability and considered tradeoffs between these three criteria using a real-time reservoir operations model. Duckstein et al. [80] [81] examined nine performance indices and using a multicriterion procedure, considered tradeoffs between these indices. Mujumdar and Vedula [215] employeded Hashimoto's reliability and resiliency risk-based performance criteria and a productivity index to examine tradeoffs between optimal operating policies.

Failure of a water supply system can have a multitude of impacts upon consumers. A domestic consumer may be only slightly inconvenienced by short interruptions to their water supply, provided these are not too frequent. In contrast, a single longer interruption to their water supply, although having the same economic consequences, may be unacceptable. An industry which is dependent on water supply may suffer considerable loss if an interruption to supply is longer in duration than some critical period. In order to accurately assess the reliability of a system it is therefore ideal to consider all consumers and to use a number of reliability indices that reflect the range of consequences of failure of the system.

2.4.4 Integrated Water Supply System Reliability Assessment

The previous sections contain a description of the major components that make up a water supply system, an identification of the need to consider the uncertainties associated with the parameters used within the water supply models, and a description of a range of reliability indices that can be used to assess the performance of the system. In this section, water supply system models developed to estimate or optimise the reliability of water supply systems will be reviewed.

Most water supply system models described in the literature that consider reliability, examine the uncertainty of reservoir inflows. Monte Carlo simulation using synthetically generated streamflow sequences and storage theory (eg. Fiering [92], Loucks et al. [190]), has been used to determine reliability estimates for a given reservoir.

Models that consider more than one of the major components of a water supply system (as described in Section 2.4.1) in the reliability assessment of the system are limited.

Methodologies for integrated reliability assessment of water supply systems have been classified into four types by Hobbs [139]. Each of these model types will be described and a review of the research undertaken in these areas will be made. The four model types are (1) contingency analysis, (2) Monte Carlo simulation, (3) closed-form analytical models, and (4) analytical simulation.

2.4.4.1 Contingency Analysis

Contingency analysis is widely used for reliability analysis of water supply systems. A system is considered 'reliable' if it can satisfy certain operating requirements under a specific set of contingency events. These events include such things as the worst drought on record, or the failure of a certain critical component in the system.

Agardy [2], Novak [225] and others highlight the need for contingency analysis for water supply systems. Having undertaken contingency analysis for the system, a contingency plan can be developed that is available for reference in the event of potential disasters occurring. It is important that these plans are both comprehensive for the water utility and simple enough to be used by the personnel operating the system in the event of such a disaster occurring.

Although the approach is relatively straight forward to apply to a water supply system, it is very difficult to assess the probability of contingency events. In contingency analysis, probability-based reliability indices are not calculated. It is therefore not possible to determine tradeoffs between cost and reliability for the system. Other disadvantages with contingency analysis are that the choices of contingencies to be considered are made arbitrarily, and different designs although able to perform satisfactorily under the specified contingencies, may have differing levels of failure probabilities for other extreme events. A system designed over a period of time using separate contingency analyses for individual components may result in over expenditure on some components and under expenditure on others. Ideally each component should be sized so that the marginal cost of improvement in system reliability is the same for each of components making up the system.

Contingency analysis is a useful tool to assess the impact of a range of contingencies on the operation of a water supply system. It is not possible, however, to use contingency analysis to determine tradeoffs between reliability and cost.

2.4.4.2 Monte Carlo Simulation

Monte Carlo simulation is a technique that can be used to calculate any desired reliability index. The technique involves the simulation of the water supply system over time, explicitly modelling inflows, demands, reservoir storage levels, restriction policies and system failures.

The approach has been presented schematically by Hobbs [139] as shown in Figure 2.6.

Random inflows, demands, component outages and other system inputs are generated using a synthetic data generation model. The response of the system is simulated in time, and as events of interest occur (eg. storage levels falling below minimum operating levels, imposition of restrictions, complete system failure etc.), statistics on system performance are accumulated. Data

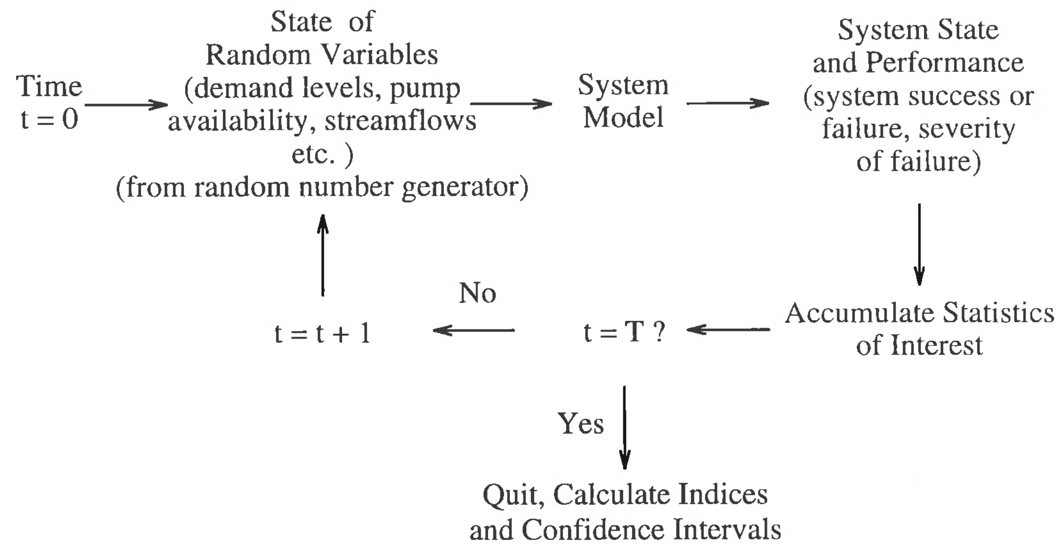


Figure 2.6: Schematic of Monte Carlo Simulation

for the calculation of reliability indices can be determined according to the requirements of the system modeller. After a given period of time, the simulation is ended and estimates of the required reliability indices are calculated.

This method is recommended by Hobbs [139] as an appropriate method for estimating the reliability of water supply systems.

2.4.4.3 Closed-Form Analytical Models

A closed-form analytical method was first proposed by Shamir and Howard [268] to determine capacity reliability. This method computed analytically the probability distribution of the chosen performance index, from the probability distributions of the random variables upon which the index depended. As noted later by Shamir [269], these computations are only feasible in relatively simple cases. Analytical expressions, if they can be obtained, do provide information regarding the dependence of the reliability performance indices on the system variables and parameters. The resulting probability distributions can be used explicitly in management models for the system.

Disadvantages of this method are that it is only useful for relatively small systems, and is impractical for design and analysis of larger systems (say with a number reservoir storages of significant capacity and a network linking these reservoirs). The probability distribution functions assumed for the behaviour of components must also be carefully selected to ensure a solution can be obtained.

2.4.4.4 Analytical Simulation

Analytical simulation is described by Hobbs [139] as 'a class of model which combines probability theory and numerical methods to calculate values of reliability indices for systems whose capacity components are subject to random outages.' In contrast to closed-form analytical methods, analytical simulation can be used to examine more complicated systems. The selection of the probability distribution functions for system components is not restricted, since numerical methods are used and the reliability indices are not expressed as explicit functions.

Analytical simulation methods have been used successfully for the analysis of bulk power system reliability as described by Billinton and Allen [19]. Bulk water and power supply systems share a number of common characteristics. These include :

1. Components such as pumps, valves and pipelines in bulk water supply systems serve the same function as power plants, substations and transmission lines in bulk power supply systems.

2. Components in both water and power supply systems are usually ar-

ranged in series and parallel.

3. Power demand and water consumption vary randomly but follow strong daily weekly and seasonal patterns and are dependent on random influences such as weather.

A major shortcoming of the analogy between bulk power supply and bulk water supply systems is that power supply systems do not have the facility to store significant amounts of power. Enhancements of the techniques applied to bulk power supply systems need to include the facility to store water in reservoirs, if they are to be successfully applied to bulk water supply systems. To date, only limited levels of storage have been included in extensions to the work undertaken in the power industry.

Analytical simulation has certain advantages over Monte Carlo simulation. The approach can be used to determine estimates of rare events and undertake parameter sensitivity analysis without resorting to long simulation runs.

Monte Carlo simulation can be used to calculate whatever reliability index is required and models can be readily developed using a multitude of available programming languages.

Packages for analytical simulation methods are not currently available.

Certain assumptions necessary in order to apply analytical simulation may be simplistic and limit the applicability of the technique to certain complex problems.

Hobbs [139] identified five analytical simulation techniques that can be applied to water supply systems. These are :

1. Loss-of-load probability (LOLP) analysis

'Loss-of-load' probability is a term obtained from the power industry and was first defined by Calabrese [26] as the 'fraction of time during which loss of load may be expected to occur during any future period'. The technique applied in the power industry was described in detail by an AIEE committee [5].

'Loss-of-load' probability analysis can be used to calculate the probability of capacity shortfalls and the level of unserved demands. The system is considered at a point in time and hence the effect of carryover storage can not be included. The total available system capacity probability distribution function is calculated by combining the failure probability distribution functions for the system's individual components. Capacity shortfall probabilities are then calculated together with the level of expected unserved demand.

2. Modified frequency-duration (FD) analysis

Frequency-duration analysis is used to estimate the probability of capacity shortfalls, together with their frequency and duration. The technique can model random demands and component outages. Streamflows can also be modelled as capacity components provided they are not inputs to storages having large capacities. The technique of frequency-duration analysis applied in the power industry does not consider storage. As noted by Hobbs [138], the direct application of this technique to a bulk water supply system will therefore provide an upper bound to the system reliability.

In order to consider the effect of storage, two modifications have been proposed by Hobbs and Beim [138]. Both modifications assume that the reservoir storage is located in the system between the capacity components and the system demand, so that capacity failures do not prevent drawdown of the reservoir.

The first modification determines upper and lower bounds to the effect of storage on the reliability calculations. These bounds, in practice, are sufficiently tight to be of use only for systems where the reservoir storages represent a small fraction of the daily demand.

The second modification calculates reliability estimates between the upper and lower bounds obtained from the first modification. This modification has been shown by Hobbs and Biem [138] to be applicable to systems with moderate amounts of reservoir storages, but still less than a single day demand.

The inclusion of reservoir storage in the application of frequency-duration analysis is limited to systems having only small storage capacities.

3. Markov chain analysis

In contrast to frequency-duration analysis, Markov chain analysis utilises a more rigorous consideration of reservoir storages. Streamflows, capacity availabilities, and system demands are modelled as Markov chains. Reservoir storages are treated as state variables with transitions between storage levels being calculated deterministically from the other system components. The Markov equations derived are solved to obtain the steady-state probabilities of reservoir storage and system failure.

Loucks et al. [190] and Houck and Cohon [145] utilise a Markov chain model representing inflows and reservoir storages as part of a reservoir system planning model. Beim and Hobbs [14] generalise the approach to include random capacity and demand. In their paper, Beim and Hobbs compared the Markov chain model with a Monte Carlo simulation model of the same system. The Markov model produced good estimates of the required reliability parameters, but computational requirements were similar for both models. Fujiwara and Ganesharajah [99] employed a Markov chain approach to assess the reliability of a water supply system comprising water treatment plant, finished water storage, pumping station and a distribution network.

4. Network reliability methods

   Network reliability analysis is used to analyse the reliability of complex networks, having several demand points. A range of methods are available and have been reviewed by Wagner et al. [309]. As networks increase in complexity, Wagner noted that 'some difficulties were encountered with the calculation of reliability measures'.

   The simpler network reliability models define failure to occur when one or more demand nodes are not connected to a supply node. The more sophisticated models also consider failure to occur when pressure at a demand node falls below a specified level.

   Certain system features are not easily represented using network reliability methods, notably storage elements, streamflows and random demands.

5. Event screening frequency-duration method

   An event screening frequency-duration method is outlined by Hobbs [139]. This method 'estimates the probability of system failure $P(F)$ for a system with dispersed demand, supply and storage'. Five steps are involved with this method :

   (a) Select appropriate exact system states.

   (b) Identify which states will lead to system failure, if all storages are empty.

   (c) Calculate the contribution each state makes to $P(F)$.

   (d) Sum each state's contribution to $P(F)$.

   (e) Calculate confidence limits for the calculated value of $P(F)$.

Both LOLP analysis and network reliability methods are limited in that it is not possible to consider storage using these techniques. In addition, random demands are not considered in network reliability methods. Markov chain analysis has none of these limitations.

Of the five analytical simulation techniques, Hobbs concludes that the modified frequency duration approach is the most appropriate analytical simulation method for the examination of reliability-cost tradeoffs for water supply systems when the capacity of storage within the system is small (less than a days demand).

For systems with storage capacities greater than a days demand, the Markov chain analysis technique is considered the most appropriate. The disadvantage with this technique is that model execution times grow dramatically with the number of system components. For systems with a large number of components, execution times using this technique can greatly exceed Monte Carlo simulation.

2.4.4.5 Comparison of Methods for Integrated Water Supply System Reliability Assessment

For water supply systems which store large amounts of raw water, representing the storage as a component whose capacity equals the natural streamflow is clearly inadequate. Reservoirs of sufficient capacity will act as balancing storages, smoothing out short-term variations between inflows and releases. The operation of the reservoir can also be modified in response to system demand and available capacity. For systems with multiple demand points, complex networks, and a large amount of storage, Hobbs and Biem [138] noted that Monte Carlo simulation is still the preferred method.

2.4.5 Summary

In this section, research in the application of risk assessment to the operation and management of water supply systems has been reviewed. Three important elements have been considered : the system model, the uncertainties involved with the estimation of parameters and the reliability indices used to assess the performance of the system. Integrated water supply system models that have been developed to estimate or optimise the reliability of water supply systems have then been reviewed. Four methodologies for the integrated reliability assessment of water supply systems have been considered including contingency analysis, Monte Carlo simulation, closed-form analytical methods and analytical simulation. These four methodologies have been reviewed and their applicability to water supply systems having a range of configurations have been compared. For water supply systems having storages larger than a days demand capacity and multiple demand points, Monte Carlo simulation is identified as the most appropriate methodology.

2.5 Context of Current Research

Research described in this thesis presents a methodology for the evaluation of reliability-cost tradeoffs for multiple reservoir headworks systems. This methodology involves the interfacing of a capacity reliability model (based on frequency duration analysis), streamflow and demand synthetic data generation models, and a headworks system simulation model.

At present no general procedure exists for the optimisation of water supply system capacity reliability. The methodology presented does not directly attempt to optimise the system reliability (although the headworks simulation model used in the case study does employ a short horizon (twelve months or less) optimisation process). Rather, the methodology can be used to explore a range of system operating rules and configurations and determine the reliability-cost tradeoffs amongst these alternatives.

Current limitations within the areas of reliability assessment are procedures for obtaining realistic estimates of reliability parameters for specific system components. Operators and designers, who are intimately involved with the ongoing operation and maintenance of these components, have a wealth of knowledge regarding failure probabilities and repair times.

An approach is presented in this thesis, that can be used to distill and enhance this knowledge by bringing a group of these experts together, using this group to identify the critical components in the system and presenting a series of 'hypothetical situations' involving the failure of one or more of these critical components. Details of this approach are described in Section 3.5.1 of Chapter 3.

2.6 Summary

In this chapter a review has been presented of the terminology that is used within the areas of risk and reliability and the application of these terms in the area of water resource systems. A broad overview of the current use of risk analysis techniques has also been considered. The application of risk assessment to the operation and management of water supply systems has been reviewed with particular reference to the three elements forming part of this assessment. These three elements are the system model, the estimation of parameters in these models, and the indices used to assess the reliability of the modelled system. Finally, the research presented in this thesis is placed within the context of previously published work.

Chapter 3

Simulation Methodology

3.1 Introduction

This chapter presents a simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir systems. The methodology considers the major elements that affect the reliability-cost performance for a water supply headworks system. These elements are :

1. The system operating rules applied by the system manager.
2. The water sources for the system.
3. The demands placed on the system by users.
4. The physical components of the system used to transfer, store and treat water.
5. The economic costs associated with the imposition of water restrictions.

During the assessment of reliability-cost tradeoffs for a water supply system, due consideration must be given to the uncertainties associated with the estimation of parameters describing each of these elements.

This chapter is structured around these five major elements. When considering the long-term performance of a water supply system, some form of simulation of the system and its operating rules is necessary. Where appropriate, mathematical optimisation can be used to assist in the selection of optimal operating strategies for the system. In Section 3.2, techniques suitable for the simulation and/or optimisation of water supply headworks systems are reviewed.

The principal water source for the majority of water supply systems is from natural catchment runoff. Inflows from these sources can exhibit considerable variation from month to month and from year to year. This variability will directly affect the reliability-cost tradeoffs for the water supply system. In Section 3.3, synthetic data generation models for inflows are reviewed.

Demands for water by consumers of a water supply system will vary according to a range of climatic and socio-economic conditions. Although variations in demand will not normally be as large as variations in water source availability, the consideration of this element in a water supply system is still important. In Section 3.4, a review is presented of techniques applicable to water demand forecasting and the use of synthetic data generation models for system demands.

Water supply headworks systems comprise physical components to transfer, store and treat water. Some of these components can be identified as critical, and failure of these components will affect the reliability-cost tradeoffs associated with the system. In Section 3.5, the 'walking party' approach is presented for the identification of critical components and the estimation of their reliability parameters. The application of the frequency-duration analysis technique is then detailed, enabling the combination of reliability parameters for multiple components. Finally, a Monte Carlo failure simulation model is presented, enabling the inclusion of component parameter reliability information in the overall reliability-cost tradeoff assessment methodology.

During periods of water shortage, it may be necessary to impose restrictions on the use of water from a water supply system. Associated with the imposition of these restrictions will be an economic loss. In Section 3.6, a review is presented of water supply restriction cost assessment methods.

Inclusion of these five major elements within an overall methodology enables the consideration of the major aspects affecting the reliability-cost tradeoffs for a water supply headworks system.

3.2 Optimisation/Simulation of Water Supply Headworks Systems

The first component of a simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems is some form of model to optimise/simulate the operation of the system. Using this model, it should be possible to consider the operation of the system under a wide range of conditions. It is important that this model represents the actual operation of the system as accurately as possible. Where models are already in use by the system operators to assist in the planning and operation of the system, it may be appropriate to adapt these models. Where no such models exist, the application of optimisation and/or simulation techniques should be adopted. In this section a review of the range of optimisation and simulation techniques is presented, together with a description of some real time operating models.

The techniques used in water resources planning for the optimisation and simulation of water supply headworks systems are many and varied. These planning techniques are concerned with the allocation and use of limited quantities of water in the 'best' possible manner so that costs are minimised and benefits to the community are maximised. In using the term 'best' there is an implication that some choice or set of alternative courses of actions is available for making the decision.

The range of planning techniques available in water resources has been described by Codner [43] and is shown in Figure 3.1.

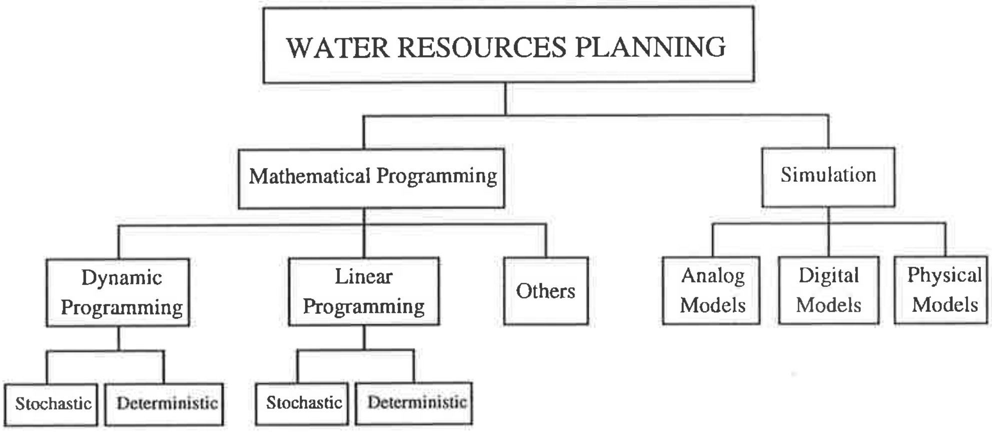


Figure 3.1: Water Resources Planning Techniques (Codner [43])

3.2.1 Review of Simulation/Optimisation Modelling Techniques

The nature of multi-reservoir water supply systems generally requires that the operational decisions made to achieve maximum benefit from the system are guided by some form of simulation or optimisation model. Optimisation models provide methods for determining the best solution to a problem from among available alternatives. These models can be applicable to planning and design as well as real time operation of the system, and are usually based on some form of mathematical programming technique.

Typical requirements for a multiple reservoir system include factors such as :

- to provide water supply for domestic, industrial and agricultural needs,
- to generate power through hydroelectric schemes,
- to provide recreation facilities,
- to provide facilities for flood mitigation and control,
- to facilitate water quality improvement,
- to provide satisfactory boating navigation facilities,
- and to provide environment enhancement for fish and wildlife.

Operational requirements are typically limited by the following constraints :

- mass conservation of water,
- physical reservoir and transfer limitations,
- the physical layout of the system,
- and contractual, legal and institutional requirements of the system.

Together with these operational constraints there are uncertainties in the hydrological terms and demands in the constraint equations. These uncertainties can be dealt with using a variety of techniques including :

- chance constrained formulations,
- stochastic programming techniques,
- and forecasting models.

Having formulated a multi-reservoir problem it is possible to reduce the problem into smaller components using such techniques as :

- Decomposition
- Partitioning
- Disaggregation
- Successive approximation

Analysis of these systems can involve many hundreds of decision variables and constraints. A review of the broad range of approaches used over the last twenty years is contained in a review paper by Yeh [328]. In this paper, the following four broad techniques are identified :

- Linear programming
- Dynamic programming
- Nonlinear programming
- Simulation

A review of optimisation techniques limited to the first two of these approachs has been presented by Crawley [48]. The review considered only two of the four approachs, as these are the most commonly used and were deemed the most appropriate for the solution of multiple reservoir headworks optimisation problems. A précis of the review undertaken by Crawley is given below.

3.2.1.1 Linear Programming

Linear programming (LP) has been widely used to solve a variety of water resource management problems. The solution of a problem using LP requires the objective function and constraint equations to be linear. Some nonlinear problems can be solved using LP through the use of linearisation, iteration or approximation techniques.

LP models are generally deterministic, however almost all hydrological data is essentially uncertain. Uncertainties in the hydrologic data can be taken into account implicitly using sensitivity analysis or by the application of a range of other techniques.

The application of LP to water resource management problems can be classified into six broad categories :

- Deterministic linear programming
- Goal programming
- Stochastic linear programming
- Successive linear programming
- Linear decision rules
- Network models

Deterministic Linear Programming involves the use of 'deterministic' or 'known' input parameters in the formulation of the LP model. The application of this technique is widespread and examples can be found in many water resource planning texts (for example Biswas [20], Loucks et al. [190]).

Goal Programming is a modified form of the standard linear programming approach and was first applied to a water resources problem by Can and Houck [27]. In contrast to the linear programming approach where penalty functions are used by the LP to differentiate between alternative feasible solutions, goal programming uses a hierarchy of goals. Attainment of the goals is sought sequentially beginning with the highest priority goal. Only when a goal is attained is any consideration given to the next lower priority goal. The method terminates when a goal that can not fully be attained is encountered. Goal programming provides an alternative approach to the application of standard linear programming when penalty or benefit functions are difficult to obtain. An extension to goal programming proposed by Changchit and Terrell [32] is entitled chance constrained goal programming (CCGP). Rather than define goals deterministically as in goal programming, CCGP defines these goals probabilistically.

Further application of goal programming to water resource problems has been presented by Loganathan and Bhattacharya [186] and Mohan and Keskar [208].

Stochastic Linear Programming considers the 'stochastic' or 'variable' nature of the input parameters in the technique. Three approaches were outlined by Crawley [48] that have been used to include these uncertainties.

- Stochastic linear programming for Markov processes

  A Markov process is one for which the probability of occurrence of a future state is dependent only on the immediately preceding state. The inclusion of these processes within a linear programming formulation has been demonstrated by Manne [197], Loucks [187], Loucks and Falkson [189], and Houck and Cohon [145].

  A shortcoming associated with the application of this approach to typical multi-reservoir systems is the level of computational effort required.

- Stochastic linear programming with recourse

  Stochastic linear programming involves two or more stages. In the first stage, an optimum solution is determined from a formulated stochastic program with recourse or chance constrained stochastic program. This first stage may be further subdivided into two or more component stages. In the second stage, a random event is assumed to have occurred and a corrective optimal solution determined.

Procedures for the solution of two-stage stochastic linear programming models have been presented by Dantzig [69], Wets [316] and Prékopa [240], Dantzig and Madansky [69], and Kall [162].

The technique has been applied to water resource problems by Dorfman [75] and Dupacova [79].

Analysis using stochastic linear programming with recourse involves the consideration of recourse action consequences for all possible outcomes of the random variables. As the size of the problem increases, the number of possible outcomes can make the solution computationally very expensive.

The effect of recourse actions is measured by a suitable estimation of loss resulting from the random variation. This measurement is difficult if not impossible and hence a further shortcoming of the method.

- Chance-constrained linear programming

  Chance-constrained linear programming (CCLP) considers the probability attributes associated with a system, and converts these attributes into equivalent deterministic constraints.

  The approach was first suggested by Charnes et al. [31] and was later applied by Re Velle et al. [248] to the operation of a multi-reservoir water supply and storage system.

  Chance-constrained formulations neither penalise explicitly, nor provide recourse action for the violation of constraints. Hogan et al. [140] argued that this limits the practical usefulness of this modelling technique.

  Another major criticism of this technique is the difficulty in determining the probability attributes of the system constraints.

Successive Linear Programming involves the successive application of a LP model to a specific problem formulation. The solution obtained from each LP formulation is used in the construction of the following model formulation. The process is continued until satisfactory convergence is achieved. Examples of the application of this technique have been presented by Grygier and Stedinger [114], Soliman and Christensen [278], Martin [199], Reznicek and Simonović [250], Tao and Lennox [287] and Dandy and Crawley [65].

The application of successive linear programming does not guarantee that a global optimum solution for the problem will be located. It is however a useful technique when the problem under consideration is larger than can practically be solved using other mathematical optimisation techniques.

Linear Decision Rules (LDR) are introduced to simplify complex programming problems and enable solutions to be more readily obtained.

ReVelle et al. [248] first suggested a LDR for the solution of a reservoir design and/or operation problem. Many researchers have since extended the work by ReVelle et al., including Loucks [188], ReVelle and Kirby [249], Nayak and Arora [219], Houck [146] and Houck and Datta [147].

Advantages of this technique are the reduction in problem complexity by the determination of chance constraints and release volumes at the beginning of the time period. Major shortcomings of this technique are the poor performance of LDR models as operating policies in water management problems, and the conservative nature of the results they produce. The use of LDR models does no guarantee the optimality of the solutions they produce.

Although LDR models have the advantage of mathematical simplicity, unless reasonable results are obtained from their application, simplicity is no justification for their use.

Network Models (also known as network flow programming models) are a special subset of linear programming. A network is constructed, consisting of a set of nodes connected by links. Associated with the flow on each link is a cost. At particular nodes in the network, a known supply or demand is applied. The basic constraints in the model represent continuity at each node. Additional constraints may be included to represent upper and lower bounds for the flow on each link. Because of the special structure of network models, the algorithms used to solve the program run much faster than general linear programming algorithms.

A reservoir system can be represented as a network with the nodes being reservoirs or demand points, and the link points being channels, streams, pipelines or reservoir carryover storage from one time period to the next. Evaporation and other losses cannot be made to depend on reservoir storage levels but must be prespecified or determined iteratively.

A network model was first applied by Evanson and Mosley [87] to evaluate and select a plan for construction and operation of a multi-basin water resource system. Beard et al. [11] highlighted that the network model presented by Evanson and Mosley would return an infeasible solution if demands could not be met in all time periods. Kuczera and Diment [171] overcame this problem through the development of a 'shortfall network'.

The application of network models to real-world water resource planning and management problems has dramatically increased with the development of a number of generalised models. These generalised models include MODSIM (Labadie et al. [174]), WASP (Kuczera and Diment [171]), WATHNET (Kuczera [173]), and REALM (Diment [73]).

As presented by Kuczera [172], network models have significant computational advantages over linear programming . Many limitations of the approach over linear programming can be overcome by approximation or iteration, while maintaining the computational advantages of the technique. Models using network programming are gaining increased popularity in the solution of certain water resource simulation and optimisation problems.

3.2.1.2 Dynamic Programming

Dynamic programming (DP) is a general optimisation technique which can be used to solve problems involving a series of steps (or stages) in time or space (Bellman [15]). The application of the technique requires the identification of a set of state variables which summarise the effects of all previous inputs to the system at any point in time (or space). As described by Hastings [131], the application of DP to an optimisation problem requires that the conditions of (1) separability and (2) optimality be satisfied.

DP utilises a recursive algorithm which is typically applied backwards in time to determine the optimum decision and its associated cost for each state, for each point in time. The method can be applied to most nonlinear objective functions meeting the conditions of separability and optimality, and is guaranteed to locate the global optimum solution.

A review of the application of DP in water resources engineering is given by Yakowitz [322]. This review details the use of DP in reservoir operation analysis, aqueduct design, irrigation system control and water quality maintenance. More recent advances in the application of DP to reservoir operations include work by Wang and Adams [312], Karamouz and Houck [164] and Karamouz et al. [165].

The principal limitation of DP is the so called 'curse of dimensionality' which effectively limits the number of state variables which can be modelled. A number of techniques have been developed to help overcome this problem of dimensionality. Some of these techniques will now be described in greater detail.

- Incremental Dynamic Programming and Discrete Differential Dynamic Programming

  Incremental dynamic programming (IDP) is a method for partly overcoming dimensionality problems associated with ordinary DP. Hall et al. [121] gave an example of the application of IDP to reservoir operations.

  Heidari et al. [132] systematised the technique and referred to it as discrete differential dynamic programming (DDDP). Nopmongcol and Askew [224] examined IDP and DDDP and concluded that DDDP was a generalised form of IDP.

  Both IDP and DDDP assume an initial trial solution to the optimisation problem. This trial solution consists of a sequence of values for the state variables (ie. reservoir storage levels) through time. DP is then used to examine variations around this policy, corresponding to each state variable being allowed to increase or decrease by one discrete level. If a new solution is found with an improved value of the objective function, this becomes the trial solution for the next iteration. The method usually converges to a local optimum, but global optimality cannot be guaranteed. The computational effort required by IDP and DDDP is considerably less than that for DP.

- Incremental Dynamic Programming with Successive Approximations

  Another method for alleviating the effects of dimensionality in dynamic programming is incremental dynamic programming with successive approximations (IDPSA). This technique has been applied to reservoir operations by a number of researchers including Larson [175], and Giles and Wunderlich [108].

  The technique breaks a DP problem which has a number of state variables, into a series of single state variable problems. A shortcoming of

IDPSA is that convergence to the global optimum solution of the original problem cannot be guaranteed. As with IDP, IDPSA reduces considerably the computational effort required to solve DP problems.

Three additional techniques are closely allied to DP. These are stochastic dynamic programming (SDP), reliability programming and the set control approach.

Stochastic Dynamic Programming (SDP) is a technique which can be used to develop optimum reservoir operating policies in the case where any particular action (eg. a reservoir release) can lead to a set of possible future states with known probabilities. Using this technique, it is possible to determine the best possible action for each state at each point in time by maximising the expected value of future benefits or by minimising expected future costs. Among researchers who have applied and extended the application of SDP to reservoir operations are Turgeon [296] [297], Stedinger et al. [281], Goulter and Tai [115], Pereira and Pinto [231], Tai and Goulter [286], Trezos and Yeh [295], Walker and Wyatt [311], Perera and Codner [233], Druce [76], Huang et al. [150], Piccardi and Soncini-Sessa [236], Braga et al. [23], Harboe et al. [128], Jain et al. [157] and Tejada-Guibert et al. [289].

The SDP algorithm is commonly used to determine a stationary operating policy using the previous period's inflow as a hydrologic state variable. The computer time required for SDP is considerably greater than that for DP and the technique is therefore only practical for systems having a limited number of reservoirs (up to three or four).

A number of variations closely allied to SDP have been proposed by researchers including the linear quadratic gaussian (LQG) control method, sampling stochastic dynamic programming (SSDP) and Bayesian stochastic dynamic programming (BSDP).

The linear quadratic gaussian (LQG) control method is an adaption of stochastic control theory. Wasimi and Kitanidis [313] in their examination of the daily operation of a reservoir system solved the DP problem with the use of a discrete-time linear quadratic gaussian (LQG) control method. A state-space model was developed for short-term forecasting of inflows into the system and the optimal releases were obtained.

Work by Georgakakos and Marks [104] introduced a new method entitled extended linear quadratic gaussian (ELQG) control using advances in stochastic control theory. If the probability distribution for future inflows can be assumed as Gaussian, and the objective function described by a quadratic, then ELQG can be used to obtain efficient solutions to much larger problems than can be solved using SDP. The limitations of ELQG are that constraints on releases and storages are not considered, consequently there is no guarantee that the optimal solutions obtained will be feasible.

Georgakakos [105] modified the ELQG control method to more effectively handle non-Gaussian storage constraints. The modifications included application of a new barrier function and the use of higher-order statistical moments when forecasting future inflows and storages. The effectiveness of the modifications was tested using a set of control and simulation experiments on the Savannah river system in the United States. Application of the ELQG control method to a six reservoir system in Arizona has been presented by Hooper et al. [142].

Sampling stochastic dynamic programming (SSDP) is a variation of SDP that was first presented by Kelman et al. [168]. This technique endeavours to capture the temporal and spatial structure of the streamflow process by considering a large number of sample streamflow sequences. SSDP determines optimal decisions by considering all streamflow scenarios simultaneously.

Karamouz and Vasiliadis [166] proposed a Bayesian stochastic dynamic programming (BSDP) model. This model used Bayesian decision theory to update the prior probabilities to posterior probabilities, reducing the impact of natural and forecast uncertainties in the model. The BSDP model was tested against two SDP models in a simulated reservoir operation and produced as good as, or better results with reduced computational effort. This work has been further extended by Vasiliadis and Karamouz [304] with the development of a demand driven stochastic dynamic programming (DDSDP) model. The DDSDP model generates operating policies taking into account monthly variation of parameters such as inflow and demand in addition to the natural and forecast uncertainty in the system. Results from the model testing highlighted that the inclusion of flow forecasts and monthly variable demands as state variables allowed the development of more efficient, realistic and robust operating policies.

Reliability Programming is based on a combination of CCLP and DP. The CCLP formulation is first solved for a range of flood and drought risk levels during each time period and the DP is then used to select between these different reliability levels. An optimal solution is obtained that will provide the highest level of reliability against drought and flood during the critical period of the year.

Application of reliability programming to the operation of reservoir systems has been described by Becker and Yeh [13], Colorni and Fronza [45], Simonović and Mariño [272], [273] [274], and Mariño and Mohammadi [198].

In a recent paper by Strycharczk and Stedinger [285], a number of serious questions were raised about the validity of results obtained using the reliability programming approach. Although reliability programming shows potential for determining optimal design and operational variables together with optimal risk strategies, doubt has been raised about the optimality of the results obtained.

The Set Control Approach has been recently presented by Georgakakos and Yao [106] [325] for the solution of reservoir system management problems. The approach assumes that the operators of a reservoir system seek to have a set of operational policies which are guaranteed to satisfy all system constraints during the period under consideration. The approach is based on DP and primarily focuses on critical periods of floods and droughts. During these periods the approach endeavours to ensure that the system constraints are not violated. System operators can then select from these operational policies a specific set that will optimise other specific system objectives.

Georgakakos [107] described the application of the set control approach to assist in the operational management of the Savannah River system. The application involved an inflow predictor, a static control module and a dynamic control module. The dynamic control module solved the stochastic control problem using the extended linear quadratic Gaussian (ELQG) control method. The essence of the method is to quantify the required operational trade-offs for the system and present the operating authority with the necessary information on which to make sound operational decisions.

3.2.1.3 Nonlinear Programming

Nonlinear programming involves the solution of programming models containing nonlinear objective functions and constraint equations. In contrast to dynamic programming, the objective function does <u>not</u> have to be separable in time. The major difficulty encountered when employing nonlinear programming is the slow convergence rates to the optimal solution and the associated levels of computational effort required.

Among the nonlinear programming techniques that have been employed are : the gradient projection method by Lee and Waziruddin [180] and Simonović and Mariño [272], the modified gradient projection method by Chu and Yeh [33], the conjugate gradient method by Divi et al.[74], the reduced gradient method by Rosenthal [255], and the application of optimal control theory.

3.2.1.4 Simulation

Simulation is a modelling technique which attempts to characterise the behaviour of a system using a computer model. Simulation primarily involves the application of a series of mathematical equations. These equations are developed to approximate the actual behaviour of the system as accurately as possible. In contrast to mathematical programming, where all possible decision alternatives are examined, simulation is limited to the consideration of a specified set of decision alternatives.

There is no certainty that simulation will obtain the optimal solution for a particular problem, and often a trial and error search process is employed to obtain a good solution. Genetic algorithm and neural network techniques are currently being examined as tools to assist this search process in problems where the solution space is large.

Since simulation may bypass optimal solutions to a problem, mathematical programming routines can be included within simulation models, to perform some form of partial optimisation. An example of this approach has been presented in a recent paper by Johnson et al. [159]. This paper considers the use of heuristic operating policies for reservoir system simulation, including the space rule described by Bower et al. [22], and New York City's 'equal-probability-of-spill' rule described by Clark [36] [37]. These rules were expressed as mathematical objective functions and were combined with constraints on the operation of the system. This produced one-period optimisation sub-models to determine releases within the overall simulation models.

Simulation has the advantage over mathematical programming that a more realistic model of the actual system (including nonlinearities and stochastic terms) can be developed.

3.2.1.5 Real Time Operating Models

The optimisation/simulation of real time reservoir operation commonly utilises forecast data. The confidence limits associated with this forecast data will widen as the forecasts extend further into the future. Real time operating models are often constructed with two sub-models : a short-term model (with horizons up to a month) and a long term model (with horizons up to a year or longer). The selection of the objective functions within these models is heavily dependent on the primary purposes for the system. In many models the constraints and objective functions are nonlinear. These nonlinearities can be effectively linearised and the resulting problems solved using linear techniques.

In all forms of operating decision models, solutions that only approximate the optimal solution can be obtained. Inherent in the practical formulation of these models are approximations of one form or another. In practical terms, the differences between solution techniques, are the speed of convergence, the computational requirements and the convenience of application.

There are many examples of the practical application of systems analysis to real time reservoir system operations. Some application to more complex systems are reviewed below.

The California Central Valley Project consists of nine major reservoirs, nine major power plants, three major canals and four major pumping plants and is managed by the Department of the Interior. Three models with differing time horizons have been developed to assist in this operation of the system. The long horizon model considers monthly steps up to a year, the medium horizon model considers daily steps up to a month, and the short horizon model considers hourly steps up to 24 hours. The system is optimised using the long horizon model, then the medium horizon model and finally the short horizon model. Outputs from the longer horizon models are used as input for the shorter horizon models. The optimisation procedure used within each model consists of an iterative LP procedure related to incremental dynamic programming with successive approximations. Further details of this model are described by Yeh [326].

The California State Water Project is intimately linked to the California Central Valley Project System described above. This project is managed by the State of California.

The primary objective of the project is to "satisfy water demands which can reasonably be served with the existing conveyance facilities". These facilities consist of seven major storages, nine major pumping plants and three major hydroelectric plants. The system is operated to produce and sell power during on-peak energy demand periods using the generating plants. During off-peak energy demand periods, power is purchased to operate the pumping plants.

A large scale simulation-optimisation model has been developed to assist in the real time operation of this system. The model can be used for yearly, weekly and daily scheduling. It comprises a suite of network flow programming, LP and simulation sub-models. Links exist between the sub-models to ensure consistency and continuity in water and power operations. Details of this project and the associated sub-models have been presented by Sabet et al. [258], Sabet and Coe [259], Chung et al. [34], and Sabet and Creel [260] [261].

The Central Arizona Project is a large aqueduct system that is used to transfer water from the Colorado River to the cities of Phoenix and Tuscon, together with water entitlements to urban, agricultural and industrial water users along its length. The project comprises approximately 220 km of aqueducts with 4 major relift pumping plants raising water a total of 142 m. The model objective is to minimise on-peak pumping while maintaining scheduled supply deliveries.

A linear programming model is used for the optimisation process and is applied to separate segments within the project. Constraint violations between segments are removed through multiple iteration. Details of this project are described by Yeh et al. [327].

The Tennessee Valley Authority Project was commenced to develop a comprehensive water resource management approach to the Tennessee Valley Authority's reservoir system. This system comprises 42 major reservoirs each with an associated power plant. The objectives of the management approach are to minimize damage due to flooding and maximise power generation while ensuring navigation requirements are met in the associated rivers.

The project is subdivided into four major segments : weekly planning, weekly scheduling, daily planning, and daily scheduling. The planning models are used for basic studies, while the scheduling models are used to assist in the day-to-day operational decision-making process. The models utilise a combination of DP and LP and are closely linked to other simulation and forecasting models. The project has been described by Shelton [271] and Wunderlich [321].

The Alcan Saguenay Hydro System is located in the Saguenay-Lac St.-Jean region of Quebec and consists of three major reservoirs, one minor reservoir and six major hydroelectric plants. A hierarchical approach was applied to the planning and operation of the system. A number of interconnected optimisation models have been developed which are linked to a real time operational model.

Four levels of optimisation are considered, comprising long-term optimisation (LTO), medium-term optimisation (MTO), short-term optimisation (STO), and real time operation (RTO). These levels use a range of optimisation techniques, including DDDP and adaptive LP. The project has been described by Unny et al. [300].

The Ottawa River Regulation Modelling System (ORRMS) was developed to allow an integrated approach to the operation of thirty three major reservoirs and forty three major hydraulic generating stations located in the Ottawa River basin. Two principal objectives have been used in the operation of the system. These objectives are to maximise energy generation and to minimise flood damage. A tradeoff curve has developed by the system operators to identify the best compromise solution between these two conflicting objectives.

A hierarchical approach based on time decomposition has been used to solve the dimensionality problem. The large initial problem has been replaced with a hierarchy of smaller problems that can be handled by the available computers. Three sub-problems were solved in series, comprising a long-term model, a mid-term model and a short-term model. The optimisation technique used within each sub-problem was LP with separable programming capabilities. Details of this project are described by Bechard et al. [12].

3.2.2 Selection of a Simulation/Optimisation Model

In the previous section, an overview has been presented of the range of simulation/optimisation modelling techniques that have been applied in the field of water resources.

The assessment of reliability-cost tradeoffs for a water supply headworks system will involve the use of some form of simulation/optimisation model. When selecting an appropriate modelling technique, the following considerations should be taken into account.

- What models are currently available ?
- Is the model applicable to the actual operation of the system ?
- What level of accuracy is required of the model in its representation of the actual operation of the system ?
- What level of precision is required of the results ?
- How flexible is the model for consideration of operating rule changes ?

An important aspect of all optimisation and simulation techniques is their applicability and ease of use in real operational situations. One of the significant conclusions drawn from the ASCE National Workshop on Reservoir Systems Operation [293] was that although considerable research effort was being expended in systems analysis, the successful application of these techniques to real reservoir operating systems was limited. The reluctance of reservoir operators to use optimisation techniques in day to day scheduling and planning were identified as :

1. Reservoir operators were not directly involved in the development of the models and hence were hesitant in their application to the real system.

2. The majority of published research dealt with simplified reservoir systems and was difficult to adapt to real systems.

3. Institutional constraints make user-research interaction difficult.

It is important therefore, in the selection of an appropriate optimisation/simulation technique, to consider the application of the developed model to the real operation of the reservoir system.

3.2.3 Summary

In this section, an overview has been provided of the available optimisation and simulation modelling techniques that have been applied to water supply systems. These techniques include the four broad areas of :

- Linear programming
- Dynamic programming
- Nonlinear programming
- Simulation

A number of examples of real time operating models for reservoir systems have also been presented.

Finally, some appropriate questions have been provided to assist in the selection of a simulation/optimisation model to be used in a reliability-cost tradeoff assessment of a water supply system.

3.3 Generation of Synthetic Inflow Data

An important component affecting the reliability-cost performance of a water supply headworks system is the water source used by the system, as described in Section 2.4.1.1 of Chapter 2. For most systems, the water source comprises natural runoff from catchments either directly collected by a reservoir, or diverted from an adjacent catchment. For some systems, this source can be supplemented by water obtained from alternative sources such as groundwater, treated stormwater, and treated sewage effluent as discussed by Clark [39].

Records for natural runoff from catchments are of finite length and will contain a limited range of extreme events. The use of these records alone, to assess the reliability-cost performance for a water supply system may yield biased results depending on the number and magnitude of extreme events contained in the record.

The second component of the methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems involves some form of model for generating synthetic inflow data. Generated data from this model can be used to examine the performance of the system using a simulation/optimisation model. In this section, a review is presented of the techniques that have been employed for the generation of synthetic inflow data.

3.3.1 Review of Generation Techniques for Synthetic Inflow Data

Data generation models aim to produce time series data which are equally likely to occur in the future as those which have occurred in the historical record. The use of these models enables the testing of proposals over a wide range of data and can provide useful information regarding possible outcomes associated with the proposals. When using stochastic data generation, the overall purpose for the generated data must be considered. The statistical properties of the data having greatest impact on the key issues under consideration will influence the type and complexity of model selected.

Baker [10] presented a detailed review of synthetic data generation modelling and a précis of this review is presented below. Baker classified synthetic data generation models into three categories :

1. Univariate Models
2. Multivariate Models
3. Multiperiod, Multivariate Models

Univariate models, were described by Baker as 'concerned with the temporal characteristics of a single time series'. Univariate models provide a foundation for data generation and utilise basic principles applied in more advanced models. Baker considered three types of univariate models : (1) autoregressive (AR) models, (2) autoregressive moving average models (ARMA) and the more general (3) autoregressive integrated moving average models (ARIMA).

Multivariate models were described by Baker as considering 'not only the time dependent nature of a series but also the spatial dependency between variates'. Principles applied in univariate models are applicable to multivariate models, although the solution for model parameters requires greater computational effort. Baker reviewed the various models described in the literature and highlighted two key papers by Benson and Matalas [16] and Matalas [201]. These papers describe research that played a key role in the development of multivariate data generation modelling.

The extension of Matalas' work on multivariate models to include multiple time periods was first proposed by Pegram and James [230]. Further details of this approach were described by Salas and Pegram [263] who presented a general multivariate multilag auto-regressive model. A model of this form is applicable to the analysis of hydrologic and water use sequences using time periods of any length.

Since the review undertaken by Baker a number of additional papers have been published in the field of synthetic inflow data generation.

Salas and Abdelmohsen [266] presented exact methods for initialising the generation of low-order multiperiod univariate and multivariate AR models and low-order multiperiod univariate and multivariate ARMA models. It was proposed that these methods should be used in preference to the more common approximate technique where a 'warm up' data set is generated and subsequently discarded.

Salas and Obeysekera [264] and Claps et al. [35] have considered the conceptual basis for the development of seasonal streamflow time series models with the aim of including physical information regarding the rainfall-runoff process in the synthetic data generation processes. Explicit relationships are established between the conceptual and stochastic parameters. Parameter estimation is undertaken at different aggregation scales using an iterative approach.

3.3.2 Application of Data Generation Models for Synthetic Inflows

In 1962, Thomas and Fiering [290] first presented an applied technique for the generation of synthetic streamflow sequences. Since that time, many synthetic hydrologic data generation models have been used in water resource planning. These include models developed by Benson and Matalas [16] [201], Young and Pisano [329], Fiering and Jackson [94] Lindner et al. [183] and Srikanthan et al. [279].

Advantages of the use of these models in water resource planning have been discussed by Fiering [92] and Fiering and Jackson [94].

Given the availability of finite and often short streamflow records, it is important to recognise that the estimated streamflow model parameters will have only limited precision. Work presented by Vicens et al. [306] detailed a Bayesian synthetic generation scheme which accounted for the parameter uncertainty due to streamflow records of limited length. In their paper, an annual univariate first-order normal autoregressive model was presented. The Bayesian approach explicitly accounted for the parameter uncertainties in the generation of synthetic streamflows. The model presented by Vicens et al. was further extended by Valdes et al. [303] to an annual multivariate first-order autoregressive model.

Work by Davis [71] and McLeod and Hipel [202] highlighted that the approach adopted by Vicens et al. and Valdes et al. only approximated the relationships between parameter uncertainty and future flow sequences. McLeod and Hipel [202] presented an alternative Bayesian method for incorporating parameter uncertainty into a simple annual streamflow generation model. Their method was structurally correct and asymptotically exact. Following the work by McLeod and Hipel, Stedinger and Taylor [280] proposed a methodology for incorporating uncertainty into the parameters of an annual streamflow model. Stedinger and Taylor's synthetic streamflow generation approach can be summarised in the following manner for a single site annual streamflow.

If V_h is defined as the vector containing the historical streamflow and V_i as the annual streamflow for the future year i then the probability density function (pdf) of V_i given V_{i-1} will be given by $f(V_i|\mathbf{\Phi}, V_{i-1})$ where $\mathbf{\Phi}$ is the parameter vector of the streamflow model. The streamflow model pdf assumes that V_i is dependent on the streamflow during the previous year $i-1$, V_{i-1}. If we further define the posterior pdf of the streamflow model parameter vector as $g(\mathbf{\Phi}|\mathbf{V_h})$, assuming $\mathbf{V}_h$ is known, then the jth future streamflow replicate can be generated in the following manner.

1. Obtain a random sample $\mathbf{\Phi_j}$ from the posterior pdf of $\mathbf{\Phi}$ as shown in Equation 3.1.

$$\mathbf{\Phi_j} \leftarrow g(\mathbf{\Phi}|\mathbf{V_h}) \tag{3.1}$$

2. Generate t years of streamflow by randomly sampling from the pdf of V as shown Equation 3.2,

$$V_{i,j} \leftarrow f(V_{i,j}|\mathbf{\Phi_j}, V_{i-1,j}) \tag{3.2}$$

for i = 1,2,......,t with $V_{0,j}$ common to all replicate sets.

Stedinger and Taylor [280] used a 50-year flow record for the upper Delaware River basin and included the uncertainties associated with three parameters : the annual mean, the annual variance and the correlation of annual flows. Stedinger and Taylor noted that to model the uncertainty in all the parameters of a monthly streamflow model would require a rather complicated analysis.

3.3.3 Summary

In order to assess reliability-cost tradeoffs for a water supply system it is important to consider a broad range of scenarios. Using synthetic streamflow data generation, catchment statistics obtained from existing historical records can be used to extend the length of the inflow data set. A range of techniques for the selection and application of streamflow data generation models have been described in the literature. For a given water supply system, an appropriate streamflow data generation model should be selected and tested against the available historical record.

3.4 Generation of Synthetic Demand Data

The third component of the simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems involves the use of a model for the generation synthetic demand data. Generated data from this model can be used as input to examine the performance of the system using a simulation/optimisation model. In this section, a review is presented of the techniques that have been employed to forecast water demands for water supply systems. Considerations affecting the selection of an appropriate model are presented and the use of the selected model for the generation of demand data described.

3.4.1 Review of Research into Water Demand Forecasting

Water demand can be divided into categories such as residential, commercial, industrial and public. In the last thirty years, a number of water demand models have been developed with the aim of forecasting demands from a set of input parameters. These models have typically been formulated to consider the effect of specific climatic and socioeconomic factors. Among the climatic and physical parameters included in these models are : rainfall, evaporation, temperature, soil water retaining properties and length of growing season. Among the socioeconomic parameters included are : water price, house and land value, land size, type of housing, density of housing, area in lawn and shrubs, disposable household income, efficiency of water-using equipment, type of metering, household size, and population size.

Maidment and Parzen [193] classified the analysis of historical municipal water use data into two approaches : (1) multiple regression and (2) time series analysis. In this review of water demand forecasting, research within each of these two categories will be considered.

3.4.1.1 Application of Multiple Regression Techniques

The application of multiple regression techniques is most appropriate when trends in water demands for a city are only slowly changing or the consideration of trends is not required. These techniques are also considered most appropriate when considering annual and monthly data. Using these techniques it is possible to correlate variations in water demand with the variables influencing this demand. A generalised linear model for water demand is given in Equation 3.3. This form of model has been termed an 'additive' model for obvious reasons.

$$Q = a_0 + a_1X_1 + a_2X_2 + \ldots + a_nX_n + e \tag{3.3}$$

where,

Q = Water demand

a_i = Regression coefficients

X_i = Variable affecting demand

e = Error term

An alternative form of this model has been termed a 'multiplicative' model, since the terms in the model are multiplied together rather than added. A generalised linear model of this form is given in Equation 3.4.

$$Q = b_0 \, (X_1)^{b_1} \, (X_2)^{b_2} \, \ldots \, (X_n)^{b_n} \, e' \tag{3.4}$$

where,

b_i = Regression coefficients

e' = Error term

It is possible to transform a 'multiplicative' model into an 'additive' model by taking logs of both sides. A model of this form is commonly referred to as a loglinear model and is given in Equation 3.5.

$$log(Q) = log(b_0) + b_1 log(X_1) + b_2 log(X_2) + \ldots + b_n log(X_n) + log(e') \tag{3.5}$$

If current water use is strongly influenced by past water use, a dynamic model which explicitly takes this influence into account may be more suitable. A dynamic model will include a term involving the water demand during the previous time period (Q_{t-1}).

Agthe and Billings [3] compared several static and dynamic multiple regression models for water demand forecasting to determine estimates of long-run marginal price elasticity of demand.

Examples of the application of multiple regression techniques to the forecasting of water demand include Howe and Linaweaver [148], Grunewald et al.

[113], Gibbs [109], Howe [149], Narayanan et al. [218], Cochran and Cotton [42], Agthe et al. [4], Carnell [28], Whitlach and Martin [317], Palencia [228], Thomas and Syme [291], Abu Rizaiza [1] and Lyman [191].

Within regression analysis it is possible to consider the time dependent nature of water use. An 'additive' and a 'multiplicative' model presented by Yamauchi and Huang [324] employed a univariate time series framework but fitted the model parameters using stepwise regression.

3.4.1.2 Application of Time Series Analysis Techniques

If water use data exhibits significant autocorrelation after trends and seasonal variation have been removed, then time series analysis is considered a more appropriate approach. Time series analysis of water use seeks to describe the characteristics of the current period in terms of events that have occurred in previous time periods.

Time series models have been developed that consider population, household income, water price, rainfall, air temperature and evaporation, and have been presented by Wong [320], Young [330], and Willsie and Pratt [318]. Climatic effects have been considered in more detail in models described by Howe and Linaweaver [148], Carver and Boland [29], and Boland et al. [21] by dividing the year into two seasons. Further division of the year into twelve months, has been considered by Morgan and Smolen [212], Katzman [167], and Cassuto and Ryan [30].

Maidment and Parzen [193] presented a univariate time series methodology entitled the cascade model. This model explicitly includes the effect of changes in socioeconomic and climatic variables on water use. This model can be used for time series analysis of monthly or weekly municipal water demand. In this model, water use is considered as the sum of a deterministic component and a stochastic component. The deterministic component considers trend and seasonality, while the stochastic component considers autocorrelation and climatic correlation. Trend, seasonality, autocorrelation and climatic correlation are progressively eliminated from the data using a cascade of four filters, leaving a random error series. This series is then analysed using a nonparametric procedure called one-sample analysis. The technique was applied by Maidment and Parzen [194] to six Texas cities, and it was demonstrated that 80% of the variation in monthly water use could be attributed to the four factors previously mentioned.

Other examples of the application of time series models to the forecasting of water demand include Danielson [67], Hansen and Narayanan [125], Hanke and de Maré [122], Maidment et al. [195], Maidment and Miaou [196], Moncur [209], Smith [277], Miaou [205] [206], and Martin and Kulakowski [200].

The application of multiple regression techniques or time series analysis for the development of a water demand model assumes that the data used for all independent variables is 'exact' and contains no errors. Kher and Sorooshian [169] presented a procedure for selecting an appropriate water demand model with its associated parameters. The procedure considers the uncertainty associated with the input data and produces a bounded solution set for the water demand model parameters.

3.4.2 Selection of a Water Demand Model

When assessing the reliability-cost tradeoffs for a water supply headworks system, it is usual to consider the tradeoffs under the current system operating rules and configuration. On the basis of assessed tradeoffs, decisions of acceptable levels of risk for the system can be made. As system conditions change, these tradeoffs will need to be reassessed, and if necessary, the decisions concerning acceptable levels of risk modified.

With respect to the demands placed by consumers on the water supply system, the primary interest is in the variability in demands caused by climatic conditions and random variations. Variations in climatic conditions are also likely to affect reservoir inflows. For example, high water demands are likely to occur during periods of low rainfall and high temperature. For this reason, it is considered beneficial if a common rainfall data set is used in both the inflow and demand forecasting models for the system under consideration.

The selection of an appropriate water demand model to be included in the examination of reliability-cost tradeoffs for a water supply headworks system, will depend on a number of factors including :

1. the availability of previously developed models,
2. the sensitivity of system reliability to demand variations,
3. the availability of necessary data,
4. the confidence limits required for the results, and
5. the ability of the model to be used to generate random demands.

In many water supply headworks systems, inflow variability will be considerably greater than demand variability. In these systems the primary concern will be the consideration of the inflow variability, with the demand variability having only a secondary impact. In these situations a simple demand model may be appropriate. In systems where demand variability is high, a more complex model will be more appropriate.

3.4.3 Generation of Synthetic Demand Data

Having selected and developed an appropriate demand forecasting model, it is necessary to use this model to generate synthetic demand data that can be used as input to the simulation model for the system. The method employed for the generation of this synthetic data will be dependent on the form of the demand forecasting model selected. Techniques for the generation of synthetic inflow data have been described in Section 3.3 of this chapter. The use of some of these techniques may also be suitable for the generation of synthetic demand data.

3.4.4 Summary

The purpose of the research described in this thesis is to develop a generalised methodology that can be used to determine reliability-cost tradeoffs for multiple reservoir headworks systems. These tradeoffs can then be used by management to select a satisfactory level of reliability and operating cost for the system being considered. Using synthetic demand data generation, demand statistics obtained from existing historical records can be used to extend the length of the demand data set.

A variety of demand models have been described in the literature. For a given water supply system, an appropriate demand model should be selected and used together with an appropriate synthetic data generation technique. It is important that the synthetically generated demand data is correlated with the synthetic inflow, as both are linked to climatic factors.

3.5 Component Reliability Analysis

Water supply headworks systems often contain components that are used to transfer water from a distant water source into reservoirs, or between reservoirs within the system. Some of these components can be identified as critical to the operation of the system. It is important that these critical components are identified and their impact on the overall reliability of the system assessed. Part of this assessment process will involve the estimation of reliability information for these components.

The fourth component of the simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems involves the use of a Monte Carlo component failure generation model. Generated data from this model can be used as input to examine the performance of the system using a simulation and/or optimisation model.

In the first part of this section, a component parameter reliability estimation methodology is presented that can be used to identify critical components in a system, and to obtain reliability estimates for these components. Having identified the critical components in a system, and obtained reliability estimates for these components, it is necessary to combine the reliability information for these individual components.

In the second part of this section, the technique of frequency-duration analysis has been described. This technique can be used to determine the reliability attributes of a set of components, combined in series, parallel or some combination of both series and parallel. Application of this technique to the assessment of reliability of bulk water transfer components in a water supply headworks system is detailed in a number of worked examples.

In the third part of this section, the application of a Monte Carlo simulation model to the generation of random failures of the bulk water transfer system is described. This model uses the results from the application of the frequency-duration analysis technique to the individual system component reliabilities.

3.5.1 Methodology for the Estimation of Component Reliabilities

Any risk analysis undertaken on a water supply system where the bulk water transfer system is considered requires some form of assessment of the reliability of the transfer system. There is increasing empirical evidence to show that the process of decomposition of a complex problem into a number of smaller problems produces greater accuracy in the results obtained (Armstrong et al. [6], MacGregor et al. [192], Hora et al. [143] [144]). If the process of decomposition is applied to the reliability assessment of a bulk water transfer system, the critical physical components in the system must first be identified. Having identified these components, estimates of the reliability parameters for these components will need to be determined.

Cullinane [54] observed that there is no comprehensive data base of reliability information for physical components and sub-components of water supply systems. Accurate determination of the reliability of these components requires knowledge of the precise reliability of the basic components and the impact on the operation of the system caused by a set of possible failures. Typical failure and repair data is available for some common components of a water supply system, but as noted by Cullinane [54], "these data will be utility specific".

A methodology for the identification of the critical components in the system and the assessment of the reliability attributes of these components is required. Having decomposed the problem into a set of subproblems, the results from these subproblems need to be combined. The technique adopted for the combining of the individual reliability attributes of critical components in the system used in this thesis is entitled 'frequency-duration analysis' and is described later in this chapter.

3.5.1.1 Factors affecting the Estimation of Parameters

As already noted, in order to include the impact of the bulk water transfer system in the overall reliability assessment for a water supply system, it is necessary to identify the critical components in the transfer system and to obtain estimates for the reliability characteristics of these components. The reliability characteristics required for these critical components are the failure frequency and the mean repair time.

In many situations, the available records of reliability information associated with the critical components in a system is limited or non-existent. In these situations, statistical analysis of the historical record is not possible. Accurate determination of the reliability of these components requires knowledge of the precise reliability of the basic components and the impact on operation of the system caused by a set of possible failures.

An alternative procedure is therefore proposed in this thesis to obtain 'best estimates' of these parameters from experts familiar with the system under consideration. As noted by Hora [144], to obtain accurate estimates of uncertain parameters from a group of experts, diligence and care must be taken in the formulation, construction and application of the proposed procedure. As a background to the methodology proposed and applied in this research work, a brief review is given of some of the processes associated with estimation of parameters of this type from individuals and groups of experts. These processes include how individuals store and recall events from memory, how individuals and groups undertake creative activities, how individuals make intuitive judgements, and how an aggregated group opinion can be obtained.

Memory

Memory is the store of information that an individual draws upon for input to a decision making process. In order for an individual to make an assessment of the failure frequency and mean repair time for a critical component in a system, it is necessary for the individual to recall from memory, instances of past failure and repair related to the specific or similar components. It is therefore helpful to consider how information is organised and stored in memory and the manner in which this information can become distorted.

Pyschologists draw the distinction between two sorts of memory ; short-term and long-term (Baddeley [7]). Short-term memory refers to that information that has recently been received and upon which decisions and actions are still being undertaken. Long-term memory refers to the repository of knowledge from both recent and distant past events. It is this long-term memory that individuals draw upon when attempting to make assessment of reliability characteristics of critical components in a system and to which this section will focus attention.

When drawing on long-term memory, psychologists generally agree that the process of recall works primarily by reconstruction (Hogarth [141]). Fragments of information are recalled that allow the reconstruction of a more complete representation of the event. These fragments are understood to be linked in a net-like collection of associations. The richer the associations, the more likely individuals are to be able to recall information about a given event. Given the limited human information-processing ability, it has been observed that individuals will use whatever cues are available to trigger these associations. These cues can include specific information presented in written, verbal or visual form in addition to the normal recall processes. Individuals cannot reassess past events in the same manner as a computer; rather fragments of information are recalled and reconstructed to form a total that is coherent to the individual.

When considering the implication of memory on the accuracy of information used in judgement and choice, it is important to understand the influences affecting the encoding, and thus selection of information, as well as the process of decoding via reconstruction. Once it is acknowledged that the ability for individuals to perceive and store information is selective rather than comprehensive, it becomes clear that certain items will carry greater 'weight' in memory than others. This weighting will affect the judgement of the individual as described in a later section. It has been observed that the process of recounting an event on several occasions can result in an individual becoming 'artificially' more certain of his or her testimony and hence ascribing greater weighting to his perception of the event. This observation has important implication for gathering evidence in decision-making situations. It has also been observed that the knowledge that an event has occurred seems to restructure memory. Our memory of the past is not a memory of uncertainties, but rather a reconstruction of past events in terms of what actually occurred.

A number of useful influences can be drawn from this brief discussion of the behaviour of human memory. Firstly, the reconstruction of events from memory can be assisted with the increased provision of cues in written, verbal and visual form. In practical terms, the physical observation of components of a system under consideration can provide additional cues to assist in memory recall together with the group discussion resulting from these observations. Secondly, the weighting ascribed to events in memory is likely to vary between individuals and can readily be distorted. It is therefore considered more appropriate to obtain a corporate assessment of past events from a range of individuals rather than from a single expert.

Creative Activity

When repairing a failed critical component in a system, there is often need for the use of creative approaches to restore the system to an operational state as quickly as possible. These approaches may require a makeshift solution to the problem to be undertaken, while a more detailed and permanent repair plan is determined. In the situation where a water supply system is in a critical condition, these makeshift repairs will be acceptable to maintain the short-term system integrity until the more detailed repair is undertaken.

As part of the assessment of mean repair times for critical components, possible creative repair solutions should be considered that may be feasible in an actual component failure event. It is therefore helpful to consider the factors associated with individual creative activity. Johnson [158] identified three stages in the creative activity of an individual :

1. Preparation

   During this stage the individual collects material and undertakes other necessary tasks such as reading or thinking about the problem in preparation for creative activity.

2. Production

   During this stage relevant ideas to the problem at hand are generated.

3. Judgement

   During this stage the ideas are evaluated prior to the selection of a 'creative' solution.

These three stages will not necessarily be undertaken in a linear fashion, but may involve revisiting at different times during the process. Work by Johnson [158] has shown that it is common for an individual to become trapped in his or her creative process by the direction of his or her own thought. What is important is that an individual be able to 'turn the problem on its head' before contemplating solutions. Motivation and attitudes have also been identified as important in the preparatory stages of creative activity. Work by Hyman [153] concluded that individuals who tend to examine the creative attempts of others in constructive rather than destructive ways probably stand more chance of generating creative, constructive solutions.

When a group creative process is adopted, Prince [241] suggests that the group should be composed of people with different levels of involvement and expertise related to the problem to be solved. Such heterogeneous groups are more likely to obtain appropriate solutions.

A range of techniques have been used for the generation of creative solutions to problems for both individuals and groups. These techniques have been described by Hogarth [141] and include :

1. Brainstorming,
2. Synetics,
3. The K-J method,
4. Morphological analysis, and
5. Cross-impacts matrices.

A number of factors can be drawn from this brief discussion on creative activity. Firstly, it is important that individuals are appropriately motivated before attempting creative activity. Individuals need to perceive the relevance and importance of the task at hand if the creativity of the individual is to be maximised. Secondly, it is important to encourage individuals to adopt a constructive rather than destructive approach to the overall process. Thirdly, if a group approach is adopted, the selection of the group should include a range of experts having knowledge of the system under consideration to enhance the overall creativity of the group.

Predictive Judgement

In order to make assessments of the failure frequencies and mean repair times for the critical components in the system, it is necessary for individuals to make predictive judgements of these reliability parameters. The mechanisms, strengths and weaknesses of the process of predictive judgement are considered in this section.

When individuals make predictive judgements, these judgements are made with reference to other information sources and cues. These sources and cues can either be seen physically or imagined at the time the judgement is made. An important aspect of judgement is the extent to which these sources and cues are available to the individual. Techniques such as 'brain storming' (Osborn [226]) and 'synetics' (Gordon [110]) endeavour to utilise the interchange of information and cues between individuals to enhance creativity and improve judgement. Some researchers assert that no conclusive evidence has been produced to confirm that the use of group techniques is more effective than individual techniques (Taylor et al. [288], Dunnette et al. [78]).

As previously noted, memory plays an important role in predictive judgement. There are many examples highlighting that the level of remembrance of an incident will significantly influence an individual's judgement. Tversky and Kahneman [298] assert that individuals use the ease with which they can recall events from memory as a factor in determining intuitive judgements of frequency. This judgemental rule they term the 'availability heuristic'. If an individual can imagine or visualize several instances of one kind of event compared with another, the individual is likely to suggest that the former has a higher frequency than the latter.

It has also been observed that the process of recounting an event on several occasions can result in an individual becoming 'artificially' more certain of his or her testimony. Given that the 'availability' in memory of information is used when making judgements of frequency, an individual having recounted an event on several occasions is likely to ascribe a higher frequency to an event than the true frequency. This observation has important implications when gathering evidence in decision-making situations.

Kahneman and Taversky [298] highlight that a major error in predictive judgement is to treat each case as unique rather that treating an event as belonging to a 'reference class' about which a lot is already known. It is important that relevant 'base-rate' data, if available, is provided to individuals before predictive judgements of a 'target' event are made.

From this brief discussion of predictive judgement, a number of influencing factors can be determined. Firstly, it is important that the level and variety of information sources or cues provided to individuals making predictive judgements is as broad as possible. The facilitation of interaction between individuals in a group together with the provision of visual stimulus through visiting the component sites is therefore recommended. Secondly, having a variety of individuals with differing background and experience is likely to enhance the accuracy of the estimates obtained, since overly conservative or optimistic estimates will be countered by individuals within the group. Thirdly, the provision of 'base-data' when available is likely to improve the accuracy of the predictive judgement of the group.

Aggregating Group Opinion

If a group assessment process is adopted then there is a need for the aggregation of the individual's assessment of reliability attributes of the separate critical components to form a group opinion.

A simple method for the aggregation of the individual's assessments is by a process of consensus. The group is encouraged to discuss their estimates of the required parameters until a group consensus is reached.

More elaborate methods have been used for the aggregation of individual's assessment including : the Delphi method (Dalkey and Helmer [56], Linstone and Tiroff [184]), the use of information theory (Pulkkinen [242]), the use of statistical science (Genest and Zidek [103]), the use of an iterative process (DeGroot [72]) and the use of bargaining theory (Weerahandi and Zidek [314]).

Having obtained the aggregated group opinion for the reliability attributes of the identified individual critical components, these results can be combined using some computational technique. Edwards et al. [83] described a method that adopted this approach entitled 'probabilistic information processing' (PIP). In this thesis, 'frequency-duration analysis' (FDA) has been used to combine the individual component reliability attributes. This technique is detailed later in this section.

Genest and Zidek [103] concluded their review of statistical aggregation techniques with the statement :

> "Well directed group interaction with unrestricted feedback would be the best approach to opinion aggregation in the context of decision making."

3.5.1.2 Important Considerations in the development of a Parameter Estimation Technique

From this brief review of the factors affecting the estimation of reliability parameters for the individual critical components of a bulk water transfer system, a number of important conclusions can be drawn that influence the formulation of an assessment technique. These include :

1. Regardless of how well the process of parameter estimation is formulated, constructed and applied, adequate documentation is paramount.

2. It is important that individuals are appropriately motivated before attempting creative activity.

3. It is important to encourage individuals to adopt a constructive rather than destructive approach to the overall process.

4. In estimating reliability parameters for components, it is helpful to call on the collective memory and experience of a group of individuals, rather than the single memory and experience of an individual.

5. The selection of the composition of the group should include a range of expertise for the system under consideration to enhance the overall creativity of the group.

6. Having a variety of individuals with differing background and experience is likely to enhance the accuracy of the estimates obtained.

7. The reconstruction of events from memory can be assisted with the increased provision of cues in written, verbal and visual form.

8. It is important that the level and variety of information sources or cues provided to individuals making predictive judgements is as broad as possible.

9. The provision of 'base-data', when available, is likely to improve the accuracy of the predictive judgement of the group.

10. For judgements that cannot be automated, awareness of possible biases and the development of good judgemental habits is important.

11. The weighting ascribed to events in memory is likely to vary between individuals and can easily be distorted.

In this next section, a methodology is presented for the identification of the critical physical components of a major water supply headworks system and the estimation of reliability parameters associated with these components. The methodology involves the dissemination of the pooled knowledge of a group of experts who have been intimately involved in the design, construction, operation or maintenance of aspects of the system. The previously mentioned factors have been considered in the formulation of this methodology.

3.5.1.3 The 'Walking Party' Approach

The approach presented in this thesis (termed the 'walking party' approach), to obtain realistic reliability assessments of critical components in a water supply headworks system, involves five distinct phases.

1. Establishment of an assessment framework.
2. Individual interviews of experts.
3. Review of the individual interview outcomes.
4. The 'walking party' process.
5. Preparation and review of the final report.

The key phase in the approach is the formation of a 'walking party'. This party comprises experts who have been intimately involved with the design, construction, maintenance or operation of the system under consideration. The term 'walking party' is used as the group physically 'walks' through the system during the assessment process. For a large system, it may be necessary for this 'walk' to be undertaken over more than one day.

Establishment of an Assessment Framework. In preparation for a realistic assessment of critical components of a water supply headworks system, a number of key questions need to be addressed in order to establish the framework for the study. These questions include :

- What are the aims and purposes of the study ?

  It is important that participants in the process are fully aware of the aims and purposes of the study. Once participants can identify the possible outcomes and potential benefits associated with the study, they will be more willing to participate and contribute.

- What is the extent of the study ?

  Boundaries for the study need to be established and made clear to the participants in both the individual interviews and the 'walking party' process. It may be necessary to reiterate these boundaries to ensure individual participants do not divert the process onto tangential issues outside the scope of the study.

- What are the assumptions adopted in the study ?

  It will be necessary for certain assumptions to be adopted for the study. For example, it may be assumed that regular on-going maintenance will continue to be performed on the components of the system, or that when a major failure occurs in the system, necessary resources will be made available to remedy the failure as quickly as possible. It is important that the major assumptions adopted during the study are clearly presented to the participants, so that their input is consistent with these assumptions.

- To what level of consequence are failure impacts to be considered ?

  It is important to establish the level of consequence of failure impacts that are to be considered during the study. For example, it may be known that failures resulting in outage times that are less than a day, will not impact significantly on the operation of the system. In this situation, only those individual components failures whose repair times exceed six hours may be selected for consideration. Allowance must be made in the selection of the minimum repair times to be considered, for the possibility of a number of independent failures occurring within a short time period. Individually, these failures may not exceed the selected consequence level, but in conjunction with other individual failures may exceed this level. Setting the level of consequence of failure impacts appropriately will help direct the attention of the participants towards critical components within the system.

- Who are the experts that need to be interviewed ?

  The design, construction, operation and maintenance of any large system will have involved a large group of experts from a diverse range of fields. It will be helpful to interview as many of these experts as possible, in order to gather the full range of available information. This information can be factual, anecdotal or simply personal opinion. Having consulted a broad group of experts, it is likely that common themes will begin to appear in the gathered information.

- Who are the key participants that should form the 'walking party' ?

  When forming the 'walking party' there are a number of considerations that need to be made. If too few participants are included in the group, then questionable opinions of certain participants may go unchallenged. If too many participants are included, then the size of the group will inhibit interaction and become unwieldy. It is recommended that the group should comprise six to ten participants together with a facilitator and scribe.

- Is there sufficient overlap of knowledge among the key participants ?

When selecting key participants, it is important to consider their mix of expertise. The inclusion of participants having some overlap of expertise will enhance discussion and ensure overly optimistic or pessimistic assessments are challenged. An appropriate mix of participants will ensure that realistic reliability parameter estimates are obtained.

Individual Interview of Experts. Those who have been intimately involved with the design, construction, operation or maintenance of a system, have a great deal of specific detail knowledge together with a 'gut feel' for how various factors will affect the performance of the system. The purpose of individual interviews with experts is to establish the range of components that are considered critical in the system and a 'feel' for the potential failure modes and repair times for these critical components. There are both advantages and disadvantages associated with individually interviewing experts. Advantages include the independent nature of information gathered and the individual's personal perspective of the system. These individual perspectives may not be obtained in a larger group, as individuals may defer to more senior and experienced participants and refrain from commenting outside their particular area of expertise. Disadvantages include the lack of interaction between participants. This interaction may result in the consideration of issues and factors that the individual participants would not otherwise have considered. Estimates and issues may also be clouded by the individual's perception of the system.

As progressive interviews are undertaken, gathered information will help guide questions raised in later interviews. Typical questions that can be asked during the individual interviews are :

- Which components make the system most vulnerable to failure ?
- What would happen if a specific component failed ?

- What spares are available in the event of a component failure ?
- How would you go about getting the system operational as quickly as possible and how long would it take if a certain event occurred ?
- How has a particular component failed in the past ?
- What effect did this component failure have on the system performance, how quickly was the component repaired, and who was involved in the repair process ?

Review of the Individual Interview Outcomes. Having interviewed as broad a range of experts as possible, it is necessary to review the information gathered. Results from this review should highlight that the performance of particular components will have limited impact on the overall system while others are critical.

Following the interview process, an inventory of the critical components can be assembled and further detailed information gathered from maintenance records, design plans and calculations, and previous analysis of these components. Possible modes of failure can be established and general requirements for their repair. Sources of spares for these components can be considered and the time required to obtain these spares.

From this inventory of critical components, details for the walking party can be prepared including specific failure scenarios to be considered.

The 'Walking Party' Process. The 'walking party' process involves the gathering of a group of experts who have broad knowledge of a particular system. Having selected the experts to form the party, and established an inventory of critical components to be assessed, these experts are taken as a group to the site(s) of the components to be considered. In close proximity to the components, the 'walking party' are presented with a range of possible scenarios. Collectively, they are encouraged to work through the issues associated with these posed scenarios. Results and issues raised during the discussion are recorded and the group is asked come to an agreement on the particular reliability estimates that are sought. When the group has reached a consensus, the next scenario is considered and the process continued.

It is important that the facilitator endeavours to keep the group focussed on the primary issues at hand. At the same time the facilitator must be sufficiently flexible to allow the group to pursue possible tangents that may provide useful additional information both inside or outside the scope of the process. It is highly recommended that in addition to the notes taken by the scribe, the discussions are recorded on audio tape. These records enable accurate review of the issues raised and discussed during the process.

Using a 'walking party' approach has a number of distinct advantages over the more common approach of interviewing individual experts or gathering the experts around a meeting room table. These advantages include :

- Raising the level of interest and enthusiasm in the process.
- Ensuring outside interruptions and distractions are kept to a minimum.
- Ensuring the group is focussed on the problem at hand.
- Ensuring the actual site conditions and limitations are considered.
- Providing additional visual stimulus regarding related components.
- The potential for issues to be raised that may not otherwise have been considered.

Disadvantages with the approach include :

- The greater time commitment required from participants than the more common approach.
- The increased possibility that the process will be distracted on to peripheral issues.
- The possibility of perception by management that the method is inefficient and the additional time requirements are unnecessary.

Preparation and Review of the Final Report. On completion of the 'walking party' process, a report detailing the results of the process should be quickly prepared and distributed for review to the participants. This will ensure discussion and issues raised during the process are still fresh in the minds of participants. Details not immediately available during the process should also be obtained as soon as possible.

Having suitably modified the report on the basis of the comments received from the participants, the document can go on to form a useful resource for future considerations of the system. Data obtained during the process can also be applied towards the reliability assessment of the system under consideration.

3.5.2 Frequency-Duration Analysis

A technique reviewed in Section 2.4.4.4 of Chapter 2, and considered applicable for the reliability assessment of bulk water transfer components of a water supply system is frequency-duration analysis. This technique is described in greater detail in the following section together with some worked examples illustrating the application of this technique to components of a bulk water transfer system.

Frequency-duration methods were first developed within the field of power system engineering to evaluate and compute reliability for systems comprising electric power generation, transmission and distribution. Hall et al. [126] described the application of this technique to parallel power generating machines. They compared the results obtained with methods used by Halperin and Adler [127], and an alternative technique discussed by Sauter et. al. [267] for twenty identical machines and twenty-two machines of varying capacities. In a later paper, Ringlee and Wood [251] applied the technique in both a power demand and capacity reserve model.

Hobbs and Beim [14] [137] adapted this technique to the analytical simulation of a water supply system. In the first of two companion papers, three versions of the frequency-duration analysis approach were presented to determine the unavailability and expected unserved demand of a water supply system. In the second paper, a Markov chain approach was also considered alongside the frequency-duration analysis approach and compared with the more realistic Monte Carlo simulation approach. This comparison verified that analytical techniques can be used to produce useful reliability indice estimates.

A difficulty encountered in the application of the frequency-duration analysis technique to water supply systems is that the effect of significant storage capacity is not easily included in the reliability calculations. The frequency-duration analysis technique will be described in detail in the following section.

3.5.2.1 The Frequency-Duration Analysis Technique

As detailed by Hobbs and Beim [137], the application of frequency-duration analysis requires that two key relationships between the mean transition rate $\lambda(A)$ for a system state A, its probability $P(A)$, its expected residence duration $E(D(A))$, and its frequency $F(A)$ are satisfied. The transition rate is the rate at which the system will leave a particular state, given the system was in that state to start with. The two key relationships that must be satisfied for the application of frequency-duration analysis are given in Equations 3.6 and 3.7.

$$\lambda(A) = 1/E(D(A)) \tag{3.6}$$

$$F(A) = \lambda(A)P(A) \tag{3.7}$$

where,

$$\begin{aligned} E(D(A)) &= \text{Expected duration of residence of system state } A \\ F(A) &= \text{Frequency of occurrence of system state } A \\ P(A) &= \text{Probability of being in system state } A \\ \lambda(A) &= \text{Mean transition rate out of system state } A \end{aligned}$$

Phillips et al. [235] noted that if the probability that an event will occur in a small-time interval is small, and if the occurrence of the event is independent of the occurrence of other events, then the time interval between occurrence of these events is exponentially distributed and the process is a Poisson process. If it is assumed that the duration in a state $D(A)$ is exponentially distributed, the instantaneous transition rate will be constant over time and hence automatically satisfy Equation 3.6. It has been demonstrated by Hobbs and Beim [137] that provided the states for individual components are statistically independent and the mean transition rates satisfy Equation 3.6, the frequency-duration analysis technique is applicable to <u>any</u> distribution of residence durations $D(A)$.

Given that Equation 3.6 is satisfied for the mean transition rates for the individual components being considered, the frequency-duration analysis technique will now be described by way of a number of worked examples, similar to those presented by Hall et al. [126], but with application to a water supply system.

Consider a single component (eg. a pump) in a water supply system. The capacity of this pump can be at its maximum rating for a period of time, changing suddenly to a reduced or zero rating when the pump fails or is taken out of service. The transition from one capacity state to another can occur at any time, and is assumed to occur instantaneously. The average amount of time that the component capacity remains at a certain rating is referred to as the mean residence time in that capacity state. The long-term average availability of a given state is the mean up time (m) for a given state divided by the mean cycle time (CT) for that particular state to occur or reoccur. Figure 3.2 demonstrates the following terms :

$$
\begin{aligned}
f &= \text{Frequency (cycles per unit time)} \\
CT &= 1/f, \text{ Mean cycle time} \\
m &= \text{Mean up time} \\
r &= \text{Mean repair time}
\end{aligned}
$$

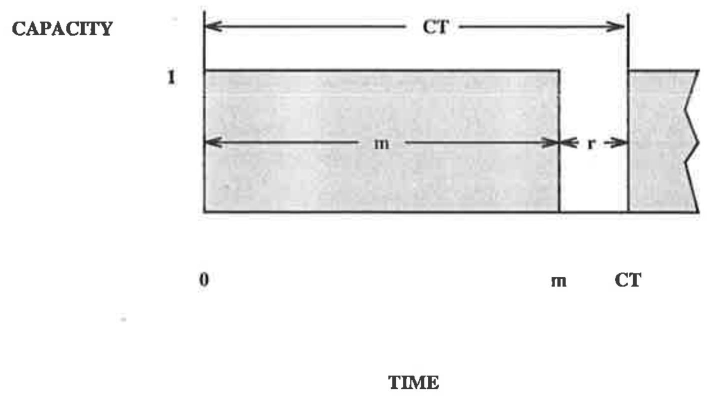


Figure 3.2: Pump Capacity vs. Time

The above definitions can also be used to define mean rates of occurrence and long-term availabilities as follows :

$$
\begin{aligned}
\lambda &= 1/m, \text{ Failure rate (failures per unit time)} \\
\mu &= 1/r, \text{ Repair rate (repairs per unit time)} \\
A &= m/(m+r) = m/CT, \text{ Availability (steady-state)} \\
\overline{A} &= 1 - A = r/CT, \text{ Unavailability (steady-state)}
\end{aligned}
$$

A two state transition diagram for a repairable pump showing the failure rate (λ) and the repair rate (μ) is shown in Figure 3.3.

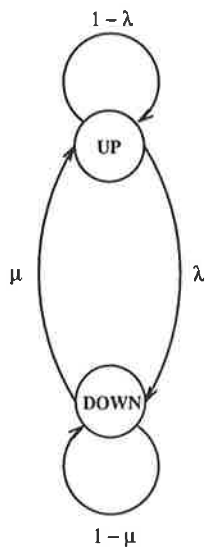


Figure 3.3: Two-State Transition Diagram for a Repairable Pump

Further definitions of the availability, transition rate and mean cycle time are given in Equations 3.8, 3.9 and 3.10.

$$\lambda = 1/(A\ CT) \tag{3.8}$$

$$\mu = 1/(\overline{A}\ CT) \tag{3.9}$$

$$f = A\lambda = \overline{A}\mu \tag{3.10}$$

Figure 3.3 presents a two-state transition diagram for a single component. The component can either be in an 'up' state or a 'down' state. Transition from the 'up' state to the 'down' state is represented by the arc labeled λ. Transition from the 'down' state to the 'up' state is represented by the arc labeled μ. The long term frequency that a state will be encountered is given in Equations 3.11 and 3.12.

$$\begin{aligned} f_{(up)} &= A\lambda && (3.11) \\ &= \text{(steady-state probability of being in a state) x (rate of departure)} \end{aligned}$$

$$\begin{aligned} f_{(up)} &= \overline{A}\mu && (3.12) \\ &= \text{(steady-state probability of not being in a state) x (rate of entry)} \end{aligned}$$

The transition rates between states can also be represented by a transition matrix where the transition rate from state i to state j is given by the matrix element of row i and column j. The two-state diagram for a repairable pump shown in Figure 3.3, can be represented by the following matrix :

$$T[i,j] = \begin{bmatrix} 1-\lambda & \lambda \\ \mu & 1-\mu \end{bmatrix}$$

3.5.2.2 Example Applications of the Frequency-Duration Analysis Technique

In order to describe the practical application of the frequency-duration analysis technique to components of a bulk water supply transfer system, it is helpful to consider a number of worked examples.

Example 1 : Two Pumps in Parallel

Consider two pumps in parallel as shown in Figure 3.4.

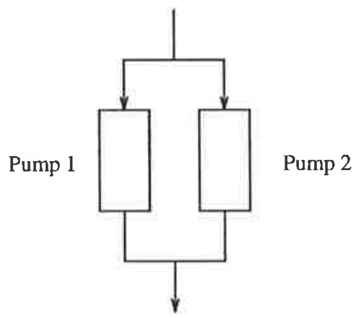


Figure 3.4: Two Repairable Pumps in Parallel

If the run-repair process for each pump is assumed to be independent, then the two pumps in parallel can be represented by the four-state transition diagram shown in Figure 3.5.

Information contained in the four-state transition diagram is also given in Table 3.1.

Alternatively, the transition rates between states for two repairable pumps in parallel can be represented by the following transition matrix :

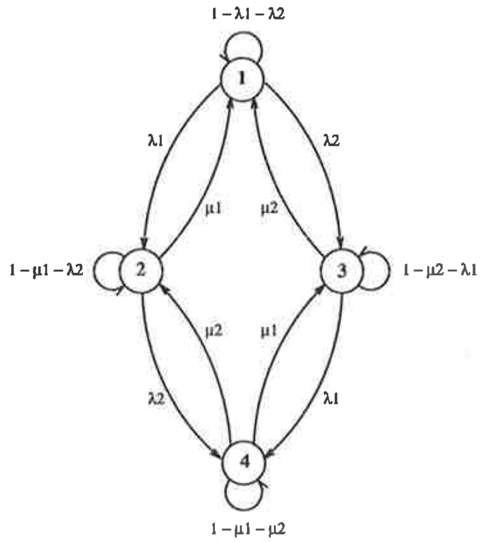


Figure 3.5: Four-State Transition Diagram for Two Repairable Pumps in Parallel

$$T[i,j] = \begin{bmatrix} 1-\lambda_1-\lambda_2 & \lambda_1 & \lambda_2 & 0 \\ \mu_1 & 1-\mu_1-\lambda_2 & 0 & \lambda_2 \\ \mu_2 & 0 & 1-\mu_2-\lambda_1 & \lambda_1 \\ 0 & \mu_2 & \mu_1 & 1-\mu_1-\mu_2 \end{bmatrix}$$

State Number	Pump1	Pump2	Rate of Departure (up)	Rate of Departure (down)
1	up	up	0	$\lambda_1 + \lambda_2$
2	down	up	μ_1	λ_2
3	up	down	μ_2	λ_1
4	down	down	$\mu_1 + \mu_2$	0

Table 3.1: Four-State Transition Table for Two Repairable Pumps

Variables given in Table 3.1 can be obtained from the above transition matrix. In the matrix, row i represents transition information concerning state i. The sum of the elements of the ith row having column numbers less than i, give the rate of departure (up) for state i. The sum of the elements of the ith row having column numbers greater than i give the rate of departure (down). The rate of entry into state i can be obtained by summing the elements in the ith column. The mean time in residence in a state can also determined from the reciprocal of the rate of departure from that state.

Consider two pumps with the failure and repair attributes given in Table 3.2.

Pump Number	Capacity (ML/day)	(f) Failure Frequency (cycles/day)	(r) Mean Repair Time (days)
1	50	0.005	2
2	100	0.005	2

Table 3.2: Example 1 : Pump Failure Frequency and Repair Attributes

Using the basic definitions previously described, the pump reliability attributes given in Table 3.3 can be determined.

Pump Number	(CT) Cycle Time (days)	(m) Mean up Time (days)	(A) Availability	(μ) Repair Rate (repairs/day)	(λ) Failure Rate (failures/day)
1	200	198	0.99	0.5	0.005051
2	200	198	0.99	0.5	0.005051

Table 3.3: Example 1 : Pump Availability and Repair Attributes (1)

With reference to the state transition diagram shown in Figure 3.5, the state pump capacities, the state availabilities, the departure rates from the state and the mean times between encountering the states can be determined as given in Table 3.4.

State Number	Available Pump Capacity	Availability	Rate of Departure (per day)	Cycle Time (days)
1	150	0.9801	$\lambda_1 + \lambda_2 = 0.010101$	1.0101 x 10^2
2	100	0.0099	$\mu_1 + \lambda_2 = 0.505050$	2.0000 x 10^2
3	50	0.0099	$\lambda_1 + \mu_2 = 0.505050$	2.0000 x 10^2
4	0	0.0001	$\mu_1 + \mu_2 = 1.000000$	1.0000 x 10^4

Table 3.4: Example 1 : Pump Availability and Repair Attributes (2)

The transition, availability and capacity information can also be written in matrix form as shown below :

$$T[i,j] = \begin{bmatrix} 0.989898 & 0.005051 & 0.005051 & 0 \\ 0.500000 & 0.494949 & 0 & 0.005051 \\ 0.500000 & 0 & 0.494949 & 0.005051 \\ 0 & 0.500000 & 0.500000 & 0 \end{bmatrix};$$

$$A[i] = \begin{bmatrix} 0.9801 \\ 0.0099 \\ 0.0099 \\ 0.0001 \end{bmatrix}; C[i] = \begin{bmatrix} 150 \\ 100 \\ 50 \\ 0 \end{bmatrix}$$

In this simplistic example, the combined capacity of the two pumps running in parallel has been assumed to equal the sum of the individual pump capacities. In practice, the state pumping capacities would need to be obtained by examining the system head versus flow curve for the particular pump configuration.

Example 2 : Two Pumps in Series

Consider the same two pumps described in the previous example, connected in series as shown in Figure 3.6.

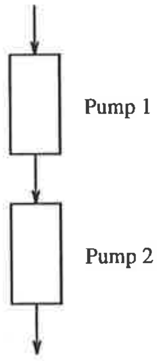


Figure 3.6: Two Repairable Pumps in Series

The four-state transition diagram shown in Figure 3.5 and the state transition table given in Table 3.1, determined for two pumps in parallel, will also apply to two pumps in series. Since the pumps are in series rather than parallel, states 2 to 4 will have an available pump capacity of zero. In a generalised form, the series capacity of two components A and B is given by Equation 3.13.

$$\text{Capacity}_{A+B} = \mathcal{F}(\text{Capacity}_A, \text{Capacity}_B) \tag{3.13}$$

where,

$\mathcal{F}()$ is the generalised function relating individual capacities to the total system capacity.

For the purpose of determining the frequency that the total pumping capacity will be zero, various states can be redefined. Cumulative states can be defined to represent the occurrence of a given pumping capacity or smaller. Considering the previous four-state transition diagram given in Figure 3.5, cumulative states 2', 3' and 4'can be defined. These new cumulative states involving the previous exact states 2, 3 and 4 are shown in Figure 3.7.

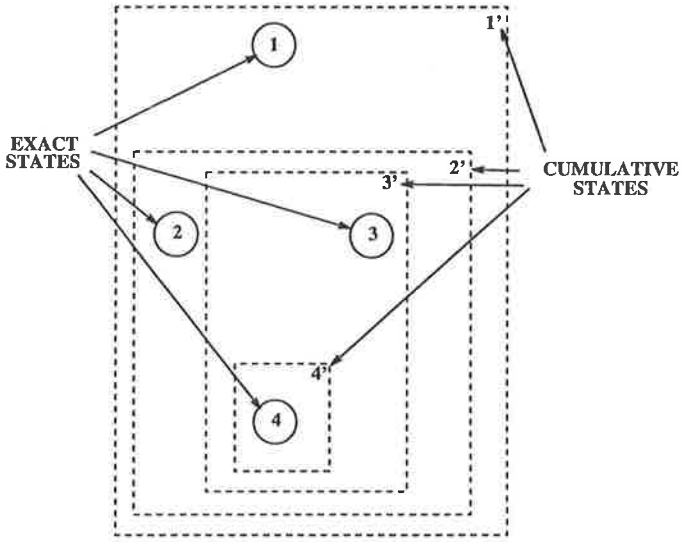


Figure 3.7: Exact and Cumulative State Transition Diagram for Two Repairable Pumps

Consideration of Figure 3.7 reveals that the frequency of encountering the new state 4' is the same as that of encountering the old state 4, as described in Equation 3.14.

$$f_{4'} \;=\; A_4(\mu_1 + \mu_2) \;=\; f_4 \tag{3.14}$$

The frequency of encountering the new state 3' is equal to the sum of the frequencies with which transfers take place from the old state 3 to the old state 1 ($A_3\mu_2$), and from the old state 4 to the old state 2 ($A_4\mu_2$). This is given in Equation 3.15.

$$f_{3'} \;=\; A_3\mu_2 + A_4\mu_2 \tag{3.15}$$

Substituting $A_4\mu_2$ from Equation 3.14 into Equation 3.15 yields Equation 3.16.

$$f_{3'} \;=\; f_{4'} - A_4\mu_1 + A_3\mu_2 \tag{3.16}$$

It is important to note that the transfers between states 3 and 4 represent failure and repair of pump 1. Since states 3 and 4 are mutually exclusive, the steady-state frequency of transfer from state 3 to state 4, and the steady-state frequency of transfer from state 4 to state 3 must be the same. The steady-state frequency is given by the product of unavailability of pump 2 and the frequency of encounter of pump 1 in an operational state as presented in Equation 3.17.

$$A_4\mu_1 = A_3\lambda_1 \tag{3.17}$$

By substituting Equation 3.17 into Equation 3.16, the recursive relationship for the frequency of encountering the cumulative state 3' will be given by Equation 3.18.

$$f_{3'} \;=\; f_{4'} - A_3\lambda_1 + A_3\mu_2 \tag{3.18}$$

In order to generalise Equations 3.14 to 3.18, the following definitions are made :

$$\begin{aligned} \lambda_{+k} &= \lambda_{up} \\ &= \text{Rate of transition out of a given} \\ & \text{capacity state } (k) \text{ to one in which} \\ & \text{more capacity is available.} \end{aligned} \tag{3.19}$$

and

$$\begin{aligned} \lambda_{-k} &= \lambda_{down} \\ &= \text{Rate of transition out of a given} \\ &\quad \text{capacity state } (k) \text{ to one in which} \\ &\quad \text{less capacity is available.} \end{aligned} \tag{3.20}$$

The frequency of encountering a state with a given capacity or less and the availability of the state is given by Equations 3.21 and 3.22. The respective transition rates are given by Equations 3.23 and 3.24.

$$f_{n'} = f_{n+1} - A_n\lambda_{-n} + A_n\lambda_{+n} \tag{3.21}$$

$$A_{n'} = A_{n+1} + A_n \tag{3.22}$$

$$(\lambda_{down})_{n'} = 0 \tag{3.23}$$

$$(\lambda_{up})_{n'} = f_{n'}/A_{n'} \tag{3.24}$$

Consider now two pumps in series having the pump availability and repair attribute data given in Tables 3.2 and 3.3.

Table 3.5 details the pump availability and repair attributes for this system.

Applying Equations 3.21, 3.22, 3.23 and 3.24, Table 3.5 can be reduced to Table 3.6, where state 3' is the cumulative state representing the combination of states 3 and 4.

State Number	Available System Capacity	Availability	Departure Rates λ_{up}	Departure Rates λ_{down}	Cycle Time (days)	Frequency of Outage
1	50	0.9801	0	0.010101	1.0101 x 10^2	0.009900
2	0	0.0099	0.500000	0.005051	2.0202 x 10^2	0.004950
3	0	0.0099	0.500000	0.005051	2.0202 x 10^2	0.004950
4	0	0.0001	1.000000	0	1.0000 x 10^4	0.000100

Table 3.5: Example 2 : Pump Availability and Repair Attributes (1)

State Number	Available System Capacity	Availability	Departure Rates λ_{up}	Departure Rates λ_{down}	Cycle Time (days)	Frequency of Outage
1	50	0.9801	0	0.010101	1.0101 x 10^2	0.009900
2	0	0.0099	0.500000	0.005051	2.0202 x 10^2	0.004950
3'	0	0.0100	0.500000	0	2.0000 x 10^2	0.005000

Table 3.6: Example 2 : Pump Availability and Repair Attributes (2)

Further application of Equations 3.21, 3.22, 3.23 and 3.24 reduces Table 3.6 to Table 3.7 where state 2' is the cumulative state representing the combination of states 2 and 3'.

State Number	Available System Capacity	Availability	Departure Rates λ_{up}	Departure Rates λ_{down}	Cycle Time (days)	Frequency of Outage
1	50	0.9801	0	0.010101	1.0101 x 10^2	0.009900
2'	0	0.0199	0.497487	0	1.0101 x 10^2	0.009900

Table 3.7: Example 2 : Pump Availability and Repair Attributes (3)

Again, this example is simplistic, as it has been assumed that the combined capacity of the two pumps running in series will equal the minimum of the individual pump capacities. In practice the state pumping capacities would be obtained by examining the system head versus flow curves. A significantly reduced, but greater than zero pumping rate may be possible when the system is operated with one pump on and one pump off. In practice, it is unlikely that the system would be operated in this manner.

Example 3 : Two equal capacity Pumps in Parallel

This third example considers two pumps configured in parallel having equal capacity. If it is assumed that the run-repair processes for each pump are independent, then the four-state transition diagram given in Figure 3.5 and the transition table given in Table 3.1 will represent the system.

If the two pumps are assumed to have the failure and repair attributes given in Table 3.8, then the application of the basic definitions previously described will result in the pump attributes shown in Table 3.9.

Pump Number	Capacity (ML/day)	(f) Failure Frequency (cycles/day)	(r) Mean Repair Time (days)
1	100	0.005	2
2	100	0.005	2

Table 3.8: Example 3 : Pump Failure Frequency and Repair Attributes

Pump Number	(CT) Cycle Time (days)	(m) Mean up Time (days)	(A) Availability	(μ) Repair Rate (repairs/day)	(λ) Failure Rate (failures/day)
1	200	198	0.99	0.5	0.005051
2	200	198	0.99	0.5	0.005051

Table 3.9: Example 3 : Pump Availability and Repair Attributes (1)

With reference to the state transition diagram shown in Figure 3.5, the state pump capacities, the state availabilities, the departure rates from the state and the mean times between encountering the states will be given by Table 3.10.

State Number	Available Pump Capacity	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9801	0	0.010101	1.0101 x 10^2
2	100	0.0099	0.500000	0.005051	2.0000 x 10^2
3	100	0.0099	0.500000	0.005051	2.0000 x 10^2
4	0	0.0001	1.000000	0	1.0000 x 10^4

Table 3.10: Example 3 : Pump Availability and Repair Attributes (2)

States 2 and 3 in Table 3.10 have identical capacities, but involve different combinations of the two pumps. With reference to Figure 3.5, there is no direct linkage between states 2 and 3. The system can only transit between states 2 and 3 if one pump fails at the same instant that the other pump is repaired. The probability of this occurring is of second-order. These two states can therefore be merged, and their frequencies and availabilities directly added. The generalised capacities, availabilities and frequencies for independent states are given by Equations 3.25, 3.26 and 3.27.

$$C_k = C_i = C_j \tag{3.25}$$

$$A_k = A_i + A_j \tag{3.26}$$

$$f_k = f_i + f_j \tag{3.27}$$

The total rates of departure to greater and lesser capacity states are given by Equations 3.28 and 3.29.

$$A_k \lambda_{up,k} \;=\; A_i \lambda_{up,i} \;+\; A_j \lambda_{up,j} \tag{3.28}$$

$$A_k \lambda_{down,k} \;=\; A_i \lambda_{down,i} \;+\; A_j \lambda_{down,j} \tag{3.29}$$

Using Equations 3.25, 3.26, 3.27, 3.28, and 3.29, the merged pump availability and repair attributes can be determined and are shown in Table 3.11.

State Number	Available Pump Capacity	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9801	0	0.010101	1.0101×10^2
2'	100	0.0198	0.500000	0.005051	1.0000×10^2
4	0	0.0001	1.000000	0	1.0000×10^4

Table 3.11: Example 3 : Pump Availability and Repair Attributes (3)

Alternatively, the problem can be analysed using a state transition matrix and capacity and availability arrays as shown below.

$$T[i,j] = \begin{bmatrix} 0.989899 & 0.005051 & 0.005051 & 0 \\ 0.500000 & 0.494949 & 0 & 0.005051 \\ 0.500000 & 0 & 0.494949 & 0.005051 \\ 0 & 0.500000 & 0.500000 & 0 \end{bmatrix};$$

$$A[i] = \begin{bmatrix} 0.9801 \\ 0.0099 \\ 0.0099 \\ 0.0001 \end{bmatrix}; C[i] = \begin{bmatrix} 200 \\ 100 \\ 100 \\ 0 \end{bmatrix}$$

The application of Equations 3.28 and 3.29 can be used to combine the elements in columns 1 and 4 of the transition matrix. By inspection, the transfer rate from state 1 to state 2' is the sum of the transfer rates from state 1 to state 2 and from state 1 to state 3. Similarly the transfer rate from state 4 to state 2' is the sum of the transfer rates from state 4 to state 2 and state 4 to state 3. This new transition matrix is shown below.

$$T[i,j] = \begin{bmatrix} 0.989899 & 0.010101 & 0 \\ 0.500000 & 0.494949 & 0.005051 \\ 0 & 1.000000 & 0 \end{bmatrix};$$

$$A[i] = \begin{bmatrix} 0.9801 \\ 0.0198 \\ 0.0001 \end{bmatrix}; C[i] = \begin{bmatrix} 200 \\ 100 \\ 0 \end{bmatrix}$$

The procedures used in Examples 1, 2 and 3 provide the necessary tools for the combination of components of a system in series or parallel and the reduction of the number of states in a system to a minimum. These procedures are the basic tools employed in the frequency-duration analysis technique. The following examples illustrate the application of the frequency-duration analysis technique to a range of pumping systems.

Example 4 : A system of Four Parallel Pumps

Consider four pumps in parallel shown schematically in Figure 3.8, having the reliability attributes given in Table 3.12.

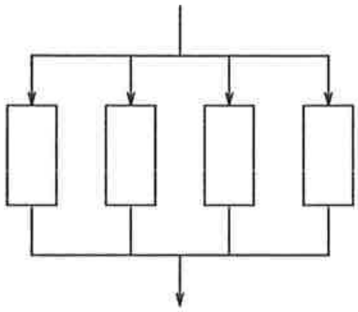


Figure 3.8: Four Parallel Pumps

Pump Number	Capacity (ML/day)	(f) Failure Frequency (cycles/day)	(r) Mean Repair Time (days)
1	50	0.005	2
2	50	0.005	2
3	100	0.005	5
4	100	0.005	5

Table 3.12: Example 4 : Pump Failure Frequency and Repair Attributes

Pump Number	(CT) Cycle Time (days)	(m) Mean up Time (days)	(A) Availability	(μ) Repair Rate (repairs/day)	(λ) Failure Rate (failures/day)
1	200	198	0.99	0.5	0.005051
2	200	198	0.99	0.5	0.005051
3	200	195	0.975	0.2	0.005128
4	200	195	0.975	0.2	0.005128

Table 3.13: Example 4 : Pump Availability and Repair Attributes (1)

The application of frequency-duration analysis techniques, produces the failure information for the overall pump system presented in Table 3.14.

State Number	Available Pump Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	300	0.93170800	0.000000	0.020357	5.2723×10^1
2	250	0.01882240	0.500000	0.015307	1.0310×10^2
3	200	0.04787490	0.201589	0.015219	9.6342×10^1
4	150	0.00096525	0.700000	0.010179	1.4588×10^3
5	100	0.00061744	0.406316	0.010062	3.8897×10^3
6	50	0.00001238	0.900000	0.005051	8.9286×10^4
7	0	0.00000006	1.400000	0.000000	1.1429×10^7

Table 3.14: Example 4 : Pump Availability and Repair Attributes (2)

Example 5 : A system of Four Parallel Pumps in Series with a single Pipeline

Consider four pumps in parallel having the reliability attributes as given in Table 3.12. In addition consider a pipeline connected in series with the pump system having failure and repair attributes given in Table 3.15. The pump and pipeline configuration is shown schematically in Figure 3.9.

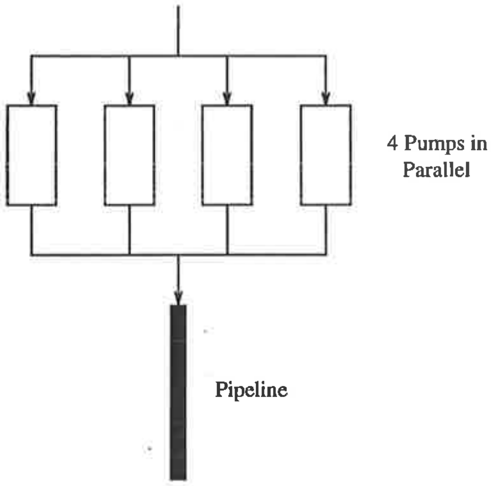


Figure 3.9: Four Parallel Pumps in Series with a Single Pipeline

Pipe Number	Capacity (ML/day)	(f) Failure Frequency (cycles/day)	(r) Mean Repair Time (days)
1	350	0.01	0.5

Table 3.15: Example 5 : Pipe Failure Frequency and Repair Attributes

State Number	Available Pipeline Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	350	0.99500000	0.000000	0.010050	1.0000 x 10^2
2	0	0.00500000	1.999950	0.000000	1.9901 x 10^4

Table 3.16: Example 5 : Pipeline Availability and Repair Attributes

The application of frequency-duration analysis techniques, produces the failure information for the overall pump system presented in Table 3.17.

Equations 3.21, 3.22, 3.23, and 3.24 can be applied to Table 3.17 to combine the zero capacity states.

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	300	0.92704900	0.000000	0.030408	3.5474 x 10^1
2	250	0.01872830	0.500000	0.025357	1.0164 x 10^2
3	200	0.04763560	0.201589	0.025270	9.2537 x 10^1
4	150	0.00096042	0.700000	0.020229	1.4457 x 10^3
5	100	0.00061435	0.406316	0.020112	3.8171 x 10^3
6	50	0.00001231	0.900000	0.015101	8.8749 x 10^4
7	0	0.00465854	2.000000	0.020357	1.0625 x 10^2
8	0	0.00009411	2.500000	0.015307	4.2244 x 10^3
9	0	0.00023938	2.201588	0.015219	1.8845 x 10^3
10	0	0.00000483	2.700000	0.010179	7.6453 x 10^4
11	0	0.00000309	2.406316	0.010062	1.3405 x 10^5
12	0	0.00000006	2.900000	0.005051	5.5633 x 10^6
13	0	0.00000006	1.400000	0.010050	1.1404 x 10^7
14	0	0.00000000	3.400000	0.000000	9.4118 x 10^8

Table 3.17: Example 5 : System Availability and Repair Attributes (1)

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	300	0.92704900	0.000000	0.030408	3.5474 x 10^1
2	250	0.01872830	0.500000	0.025357	1.0164 x 10^2
3	200	0.04763560	0.201589	0.025270	9.2537 x 10^1
4	150	0.00096042	0.700000	0.020229	1.4457 x 10^3
5	100	0.00061435	0.406316	0.020112	3.8171 x 10^3
6	50	0.00001231	0.900000	0.015101	8.8749 x 10^4
7'	0	0.00500006	1.999992	0.000000	9.9999 x 10^1

Table 3.18: Example 5 : System Availability and Repair Attributes (2)

The combined states are shown in Table 3.18. State 7' in Table 3.18 is a cumulative state representing the states 7 to 14 given in Table 3.18.

Comparison of Table 3.18 with Table 3.13 shows that the occasional unavailability of the pipeline has a dramatic impact on the cycle time for the zero system capacity state.

Example 6 : A system of Four Parallel Pumps in Series with a system of Two Parallel Pumps

The use of state transition tables in this example, was not possible, since specific information regarding the state transfer rates to specific states had been eliminated during the reduction process. Using state transition matrices, information pertaining to individual transfer rates between specific states was still available. The use of state transition matrices was therefore adopted in the application of the technique to the metropolitan Adelaide bulk water transfer system.

Consider a set of four pumps in parallel, having reliability attributes given in Table 3.12 and a set of two pumps in parallel, having reliability attributes given in Table 3.8. The pump configuration is shown schematically in Figure 3.10.

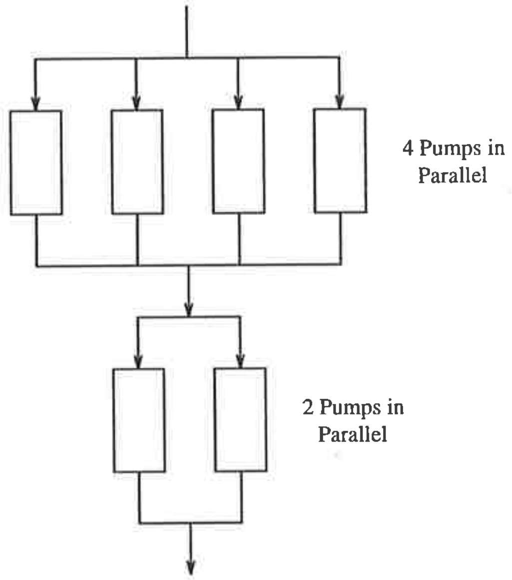


Figure 3.10: Four Parallel Pumps in Series with Two Parallel Pumps

If these two sets of pumps are linked in series, the application of frequency-duration analysis techniques, produces the failure information for the overall pump system presented in Table 3.19. States 1 to 7 are obtained by combining state 1 of the two pump sub-system with states 1 to 7 of the four pump sub-system. States 8 to 14 are obtained by combining state 2 of the two pump sub-system with states 1 to 7 of the four pump sub-system. States 15 to 21 are obtained by combining state 3 of the two pump sub-system with states 1 to 7 of the four pump sub-system.

Examination of Table 3.19 reveals that the transition between states 2 and 3, and state 1 does not involve a change in system capacity. Similarly the transition between state 12 and state 5 does not involve a change in system capacity.

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9131670	0.000000	0.030458	3.5954 x 10^1
2	200	0.0184478	0.500000	0.025408	1.0317 x 10^2
3	200	0.0469222	0.201589	0.025320	9.3923 x 10^1
4	150	0.0009460	0.700000	0.020280	1.4675 x 10^3
5	100	0.0006052	0.406316	0.020163	3.8747 x 10^3
6	50	0.0000121	0.900000	0.015152	9.0093 x 10^4
7	0	0.0000000	1.400000	0.010101	1.1577 x 10^7
8	100	0.0184478	0.500000	0.025408	1.0317 x 10^2
9	100	0.0003727	1.000000	0.020357	2.6297 x 10^3
10	100	0.0009479	0.701589	0.020270	1.4614 x 10^3
11	100	0.0000191	1.200000	0.015229	4.3056 x 10^4
12	100	0.0000122	0.906316	0.015112	8.8773 x 10^4
13	50	0.0000002	1.400000	0.010101	2.8943 x 10^6
14	0	0.0000000	1.900000	0.005051	4.2418 x 10^8
15	0	0.0000932	1.000000	0.020357	1.0519 x 10^4
16	0	0.0000019	1.500000	0.015307	3.5061 x 10^5
17	0	0.0000048	1.201589	0.015219	1.7166 x 10^5
18	0	0.0000001	1.700000	0.010179	6.0579 x 10^6
19	0	0.0000000	1.406317	0.010062	1.1435 x 10^7
20	0	0.0000000	1.900000	0.005051	4.2418 x 10^8
21	0	0.0000000	2.400000	0.000000	6.6667 x 10^{10}

Table 3.19: Example 6 : System Availability and Repair Attributes (1)

Table 3.20 can be obtained by reordering the states shown in Table 3.19.

States having capacity equal to zero in Table 3.20 can be combined to produce the state transition table given in Table 3.21.

Table 3.21 can be further reduced by combining states having the same capacities that are greater than zero. The reduced state transition table is given in Table 3.22.

In this example, the $2^6 = 64$ possible combinations of six pumps in either

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9131670	0.000000	0.030458	3.5954×10^{1}
2	200	0.0184478	0.500000	0.025408	1.0317×10^{2}
3	200	0.0469222	0.201589	0.025320	9.3923×10^{1}
4	150	0.0009460	0.700000	0.020280	1.4675×10^{3}
5	100	0.0006052	0.406316	0.020163	3.8747×10^{3}
6	100	0.0184478	0.500000	0.025408	1.0317×10^{2}
7	100	0.0003727	1.000000	0.020357	2.6297×10^{3}
8	100	0.0009479	0.701589	0.020270	1.4614×10^{3}
9	100	0.0000191	1.200000	0.015229	4.3056×10^{4}
10	100	0.0000122	0.906316	0.015112	8.8773×10^{4}
11	50	0.0000121	0.900000	0.015152	9.0093×10^{4}
12	50	0.0000002	1.400000	0.010101	2.8943×10^{6}
13	0	0.0000000	1.400000	0.010101	1.1577×10^{7}
14	0	0.0000000	1.900000	0.005051	4.2418×10^{8}
15	0	0.0000932	1.000000	0.020357	1.0519×10^{4}
16	0	0.0000019	1.500000	0.015307	3.5061×10^{5}
17	0	0.0000048	1.201589	0.015219	1.7166×10^{5}
18	0	0.0000001	1.700000	0.010179	6.0579×10^{6}
19	0	0.0000000	1.406317	0.010062	1.1435×10^{7}
20	0	0.0000000	1.900000	0.005051	4.2418×10^{8}
21	0	0.0000000	2.400000	0.000000	6.6667×10^{10}

Table 3.20: Example 6 : System Availability and Repair Attributes (2)

'up' or 'down' states can be reduced to 5 capacity states. As the number of components and the number of states per component increases, the application of frequency-duration analysis can dramatically reduce the total number of states that need to be considered for the overall system.

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9131670	0.000000	0.030458	3.5954×10^1
2	200	0.0184478	0.500000	0.025408	1.0317×10^2
3	200	0.0469222	0.201589	0.025320	9.3923×10^1
4	150	0.0009460	0.700000	0.020280	1.4675×10^3
5	100	0.0006052	0.406316	0.020163	3.8747×10^3
6	100	0.0184478	0.500000	0.025408	1.0317×10^2
7	100	0.0003727	1.000000	0.020357	2.6297×10^3
8	100	0.0009479	0.701589	0.020270	1.4614×10^3
9	100	0.0000191	1.200000	0.015229	4.3056×10^4
10	100	0.0000122	0.906316	0.015112	8.8773×10^4
11	50	0.0000121	0.900000	0.015152	9.0093×10^4
12	50	0.0000002	1.400000	0.010101	2.8943×10^6
13	0	0.0001000	1.000250	0.000000	9.9913×10^3

Table 3.21: Example 6 : System Availability and Repair Attributes (3)

State Number	Available System Capacity (ML/day)	Availability	Departure Rates		Cycle Time (days)
			λ_{up}	λ_{down}	
1	200	0.9785370	0.000000	0.011024	9.2700×10^1
2	150	0.0009460	0.700000	0.020280	1.4675×10^3
3	100	0.0204049	0.496922	0.005210	9.7600×10^1
4	50	0.0000124	0.900000	0.005151	8.9285×10^4
5	0	0.0001000	1.000250	0.000000	9.9913×10^3

Table 3.22: Example 6 : System Availability and Repair Attributes (4)

Example 7 : Analysis of Pump Stations in Series having Interchangeable Components

In this example, components comprising a pumping system are assumed to be fully interchangeable, and the impact of this assumption on the overall system reliability is examined.

Consider a pumping system comprising three pump stations in series. Within each pump station, three pumps operate in parallel. This system is shown schematically in Figure 3.11.

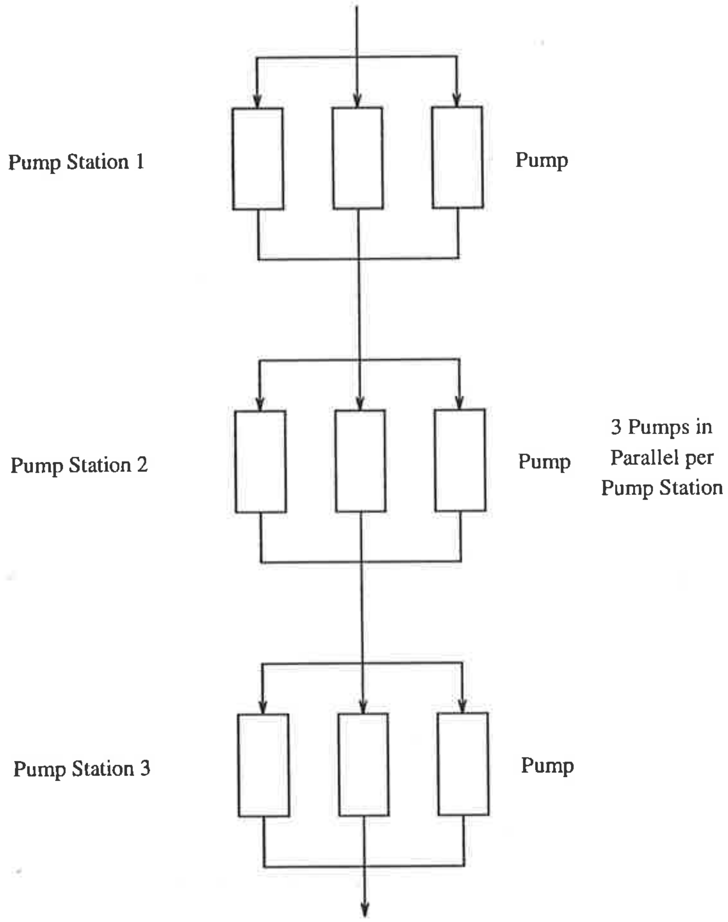


Figure 3.11: Three Pump Station Schematic

The nine pumps in the system are assumed to be identical and fully interchangeable (eg. a pump at any pump station is interchangeable with any other pump in the system). The time taken to relocate a pump is also assumed to be sufficiently small to be negligible. This interchangeability of these components can be shown to have a significant effect on the overall pump system reliability.

If a single pump fails at a particular pump station, then the capacity of the system will be reduced to $\frac{2}{3}$. If however a second pump fails, the system capacity will not be further reduced. If the second pump fails at a different pump station then the system capacity will be unaffected since each pump station will still have at least $\frac{2}{3}$ of the full pumping capacity available. If the second pump fails at the same pump station, then, since the pumps are identical and fully interchangeable, a pump from one of the other two fully operational pump stations in the system can be relocated to replace the second failed pump. If a third pump fails, the system capacity can still be maintained at $\frac{2}{3}$ full capacity. If four pumps fail, the system capacity is reduced to $\frac{1}{3}$. Table 3.23 presents the number of operational pumps and the corresponding pumping system capacity when individual pumps are fully interchangeable.

Operational Pumps	System Capacity (%)
9	100
8	66.6
7	66.6
6	66.6
5	33.3
4	33.3
3	33.3
2	0.0
1	0.0
0	0.0

Table 3.23: Pipeline System Capacity with Interchangeable Pumps

In contrast, if the pumps are non-interchangeable, then the overall system capacity is shown in Table 3.24.

Operational Pumps in Pump station 1	Operational Pumps in Pump station 2	Operational Pumps in Pump station 3	Pipeline System Capacity (%)
3	3	3	100
2	3	3	66.6
3	2	3	66.6
2	2	3	66.6
3	3	2	66.6
2	3	2	66.6
3	2	2	66.6
2	2	2	66.6
1	Any $\geq$= 1	Any $\geq$= 1	33.3
Any $\geq$= 1	1	Any $\geq$= 1	33.3
Any $\geq$= 1	Any $\geq$= 1	1	33.3
0	Any	Any	0
Any	0	Any	0
Any	Any	0	0

Table 3.24: Pipeline System Capacity with Non-interchangeable Pumps

A comparison of Tables 3.23 and 3.24 reveals that in the event of multiple pump failures, the system capacity is greatly increased by the ability to interchange pumps between pump stations in the system.

The difficulty arises when attempting to quantify this benefit using frequency-duration analysis. Although the pump stations are in series and the pumps within the pump stations are in parallel, when pumps are interchangeable the separate pump stations are <u>not</u> in fact independent. The following steps can be used to analyse a system of this nature.

1. Consider each of the pump stations separately, and determine reliability details for the pumps in parallel within the pump station using frequency-duration analysis.

2. Apply frequency-duration analysis to two of the pump stations in series, however do <u>not</u> compress the resultant states.

3. Consider the resultant uncompressed states and combine only those states with the same number of 'failed' and 'operational' pumps. (There will be a number of states having the same capacity.)

4. Using frequency-duration analysis, combine the results obtained in step 3 with the next pump station in series. Do <u>not</u> compress the resultant states.

5. Repeat steps 3 and 4 until all pump stations have been considered.

This procedure has been applied to the pump station configuration shown schematically in Figure 3.11. For each of the nine pumps in the pumping system, the reliability data given in Table 3.25 is assumed.

Capacity (ML/day)	(f) Failure Frequency	(r) Mean Repair Time (days)
100.0	0.0006667	15.0

Table 3.25: Example 7 : Pump Failure Frequency and Repair Attributes

If the pumps are assumed to be completely independent and non-interchangeable then the application of frequency-duration analysis produces the results shown in Tables 3.26 and 3.27.

If the same system is considered with interchangeable pumps then the results shown in Tables 3.28 and 3.29 are obtained from the application of the frequency-duration analysis.

Examination of the results from the preceding example highlight a number of interesting points. When interchangeable pumps are used in the pump-

State	System Capacity (ML/day)	Rate of Entry	Rate of Departure (up)	Rate of Departure (down)
1	300.0	0.064687	0.000000	0.006061
2	200.0	0.139354	0.064687	0.001387
3	100.0	0.201387	0.133294	0.000674
4	0.0	0.000674	0.200000	0.000000

Table 3.26: Example 7 : Pipeline System Reliability Information with Non-interchangeable Pumps (1)

State	State Probability	State Mean Duration (days)	Cycle Time (days)
1	0.9135×10^{0}	165.000	1.8062×10^{2}
2	0.8559×10^{-1}	15.135	1.7683×10^{2}
3	0.8907×10^{-3}	7.465	8.3802×10^{3}
4	0.3000×10^{-5}	5.000	1.6667×10^{6}

Table 3.27: Example 7 : Pipeline System Reliability Information with Non-interchangeable Pumps (2)

State	System Capacity (ML/day)	Rate of Entry	Rate of Departure (up)	Rate of Departure (down)
1	300.0	0.064019	0.000000	0.006061
2	200.0	0.270043	0.064019	0.000004
3	100.0	0.465494	0.263982	0.000000
4	0.0	0.000000	0.465490	0.000000

Table 3.28: Example 7 : Pipeline System Reliability Information with Inter-changeable Pumps (1)

State	State Probability	State Mean Duration (days)	Cycle Time (days)
1	0.9135×10^{0}	165.000	1.8062×10^{2}
2	0.8648×10^{-1}	15.619	1.8061×10^{2}
3	0.1210×10^{-5}	3.788	3.1296×10^{6}
4	0.3537×10^{-12}	2.148	6.7032×10^{12}

Table 3.29: Example 7 : Pipeline System Reliability Information with Interchangeable Pumps (2)

ing system considered, the following conclusions can be drawn concerning the pumping system reliability attributes :

1. There is no change in the state probability for state 1. The consequence of any pump failing will automatically reduce the system capacity whether pumps are interchangeable or not.
2. The probability of being in state 2 increases marginally.
3. The probability of being in state 3 is reduced by a factor of approximately 700.
4. The probability of being in state 4 is reduced by a factor of approximately 10^6.

The impact on the probabilities for states 2, 3 and 4 is significant on the operation of a system. The probabilities of having zero pumping capacity is greatly reduced, ensuring that at least a portion of the pumping capacity is available for operation of the system during the majority of time.

The preceding examples have detailed the practical application of frequency-duration analysis technique to components of a bulk water supply transfer system. The examples highlight the recursive technique that can be used to combine the reliability attributes of components in series and parallel and to reduce the number of capacity states to a minimum. In the following section, the use of Monte Carlo simulation is presented for the generation of random capacity failures of a bulk water supply transfer system.

3.5.3 Monte Carlo Failure Generation Model

Having determined the failure attributes of the combined components using the frequency-duration analysis technique, the impact of this information needs to be included within the overall reliability assessment of the headworks system.

If it is assumed that the transition rate between states is constant over time, then Equation 3.6 will be automatically satisfied as noted by Hobbs and Beim [137]. If it is assumed that the remaining two conditions given in Equations 3.6 and 3.7, then the application of the frequency-duration analysis technique has been shown to be valid.

Since it has been assumed that the transition rates between states are constant over time, the transitions between states for a given time period can be determined by selecting uniformly distributed random numbers in the range 0 to 1. For the case of a two-state system, if the generated random number is greater than or equal to the transition probability obtained from the transition matrix, then a transition is assumed to have occurred. If the random number is less than the transition probability, no transition is assumed to have occurred.

The use of a random number generator forms the basis for a Monte Carlo simulation model. When developing a model of this form, it is critical that a 'truly random' random number generator is employed. When selecting the random number generator, the required length of generated record must be considered to ensure that the generator does not produce cyclical patterns within the record length required. The following procedure is proposed for generating the random failure events.

1. Assume the system starts in state 1 at the commencement of the Monte Carlo generation process.

2. Generate a random number in the range $0 \rightarrow 1$ based on the assumed combined component failure distribution.

3. Look up the row for the initial state in the transition matrix.

4. Proceed across the row until the next element in the transition matrix will make the accumulated transition matrix values exceed the generated random number.

5. Determine the final state for the time period.

6. Increment the time counter by 1 and set the initial state for the next time period equal to the final state of the previous time period.

7. If the required length of failure record has not been generated proceed to step 2.

3.5.4 Procedure for Component Reliability Analysis

The following procedure is proposed for the inclusion of component reliability in the reliability-cost assessment of a water supply headworks system.

1. Identify the critical components and obtain individual reliability attributes for these critical components using the component parameter reliability estimation methodology described in Section 3.5.1.

2. Apply frequency-duration analysis to combine the reliability attributes of the individual components and produce frequency and duration information for the combined components.

3. Generate random failure events using a Monte Carlo simulation model based on the combined component reliability information.

4. Assess the impact of the random component failures using a simulation model of the headworks system, together with synthetically generated inflow and demand data.

3.5.5 Summary

Pumps and pipelines are often used to transfer water within water supply headworks systems. These components have finite lives and are subject to unpredictable failures.

In systems where these components play an important role in the operation of the system, the estimation of the component parameter reliability attributes is critical in the assessment of the overall system reliability. A methodology has been presented for the estimation of these parameters which is applicable to any complex system. The approach has a number of advantages over the use of general component reliability data or the more conventional approach of gathering experts around a table.

Having determined realistic estimates for the individual component parameter reliability attributes, it is beneficial to be able to combine and simplify these parameters. Using a technique known as frequency-duration analysis it is possible to simplify the reliability attributes for a set of components. The background and development of this recursive technique has been described and its application to the reliability assessment of a set of components in series and parallel has been illustrated with a number of practical examples. The technique assumes that the states of the individual components are statistically independent and the mean transition rates are defined as the reciprocal of the mean duration.

Finally, a Monte Carlo simulation model is described that can be used to generate sequences of random state occurrences and durations, using the results of the frequency-duration analysis for the combined system components.

3.6 Water Supply System Restriction Costs

The fifth component of the simulation methodology, involves the consideration of the economic costs associated with the application of water supply restrictions.

During the operation of a water supply system, there will be periods when the likelihood of the system running out of water increases, for example when the system is exposed to drought conditions. During these periods, it is common for water supply authorities to endeavour to reduce demand to ensure supply can be maintained until the system reaches a more satisfactory condition. A number of approaches have been employed by water authorities for this purpose including :

- the implementation of voluntary water restrictions,
- the implementation of mandatory water restrictions, and
- the use of pricing policy.

Implementation of the first two of these approaches will normally result in economic loss to both consumers and the water supply authority. The use of pricing policy will result in a net economic loss if the current water price is set at the social optimum.

In this section, a review is presented of the range of approaches that have been used to the determine the economic costs associated with water supply system restrictions and an appropriate method selected.

3.6.1 Review of Water Supply System Restriction Cost Assessment Methods

Most water supply authorities follow a multi-stage restriction program where successive stages are implemented sequentially as the risk of undesirable events increases. For example, a survey of water rationing policies for major urban headworks systems in Australia, undertaken by Sheedy and Kesari [270], highlighted the widespread use of both voluntary and mandatory multi-stage water rationing.

From an economic point of view, Moncur [209] [210] suggested the use of pricing policy as the most efficient approach to the rationing of water during periods of supply shortage. In practice, few water authorities have employed this approach as practical and political considerations are considered to outweigh the associated efficiencies.

An indirect approach for the determination of water supply restrictions costs is to identify the incremental cost of increasing the supply capacity so that water restrictions are no longer necessary. This approach was applied by Boland et al. [21] to the water supply system of the Washington Suburban Sanitary Commission. A similar approach was proposed by Clarke et al. [40] and applied to the supply system of the Anglian Water Authority.

Dziegielewski and Crews [82] presented a framework for water supply operators to formulate least-cost drought emergency plans. Within this framework, tradeoffs between the expected value of the long-term cost of coping with water supply deficits and the cost of the long-term water supply/conservation projects were examined. Local economic losses caused by water restrictions were assumed to comprise :

- loss of consumer surplus,
- loss of profit by local industry,
- loss of production payroll for externally owned industry, and
- loss of water revenue from externally owned industry.

Work undertaken by Dandy [66] has considered the assessment of the economic losses resulting from the rationing of outdoor water use. In this work, the economic cost of water restrictions were assumed to equal the loss of consumer surplus plus the loss of producer surplus. The external costs of restrictions due to 'browning' of the city were not considered. A general expression for the economic costs of time restrictions on outdoor water use was presented.

3.6.2 Proposed Methodology for the Assessment of Water Supply System Restriction Costs

For many water supply systems, a significant proportion of the consumer demand during summer months consists of outdoor water use. For example, in the metropolitan Adelaide system, this proportion can be as high as 70% as noted in an Engineering and Water Supply Department of South Australia (EWS) report [90]. The need for the imposition of water restrictions is also likely to coincide with these periods of high outdoor water use.

If the imposition of outdoor water restrictions is the mechanism adopted by a water supply authority to reduce demand during periods of water shortage, then the approach proposed by Dandy [66] is considered appropriate for the assessment of the economic costs associated with the implementation of these restrictions.

If households are assumed to have demand functions of constant elasticity not equal to -1, then Equation 3.30 describes the economic loss associated with outdoor water use restrictions.

$$\begin{aligned} L &= p\, Q_o \left\{ \left(\frac{\overline{\varepsilon}}{1+\overline{\varepsilon}}\right) [1-(1-\overline{r})^{(1+\overline{\varepsilon})/\overline{\varepsilon}}] \right. \\ & \left. - \frac{1}{2\overline{\varepsilon}}(1-\overline{r})^{(1-\overline{\varepsilon})/\overline{\varepsilon}}\, S_r^2 \;+\; (1-\overline{r})^{1/\overline{\varepsilon}}\, \rho_{qr}\, V_q\, S_r \right\} \end{aligned} \qquad (3.30)$$

where,

L = Economic loss (\$/unit time).

p = Water price (\$/m^3).

Q_o = Total unrestricted outdoor water demand at price p (m^3/unit time).

$\overline{r}$ = Mean fractional reduction in household water demand due to restriction imposition (%).

$\overline{\varepsilon}$ = Mean price elasticity of demand for outdoor water use for all households.

S_r = Standard deviation of fractional reduction in individual household water demands due to restriction imposition.

ρ_{qr} = Cross correlation coefficient between individual outdoor household water consumptions and fractional reduction in individual household water demands due to restriction imposition.

$$V_q \;=\; \text{Coefficient of variation of individual household outdoor water demand functions.}$$

This equation assumes that there is little variation in the price elasticity of demand for individual households. When the standard deviation of fractional reduction in individual household water demands due to restriction imposition (S_r) is small, Equation 3.30 reduces to Equation 3.31.

$$L \;=\; p\, Q_o \left(\frac{\overline{\varepsilon}}{1+\overline{\varepsilon}}\right) [1-(1-\overline{r})^{(1+\overline{\varepsilon})/\overline{\varepsilon}}] \tag{3.31}$$

For varying levels of water restrictions, the total economic loss can be calculated from the above equation and used in the consideration of reliability-cost tradeoffs for a water supply system.

If it is not possible to estimate the price elasticity of household demand for outdoor water use, a simpler approach may be necessary for the determination of economic losses. In this approach, the cost of restrictions is assumed to equal the loss of producer surplus as given in Equation 3.32.

$$L = p\, Q_o\, \overline{r} \tag{3.32}$$

This simpler approach does not consider the loss of consumer surplus resulting from water restrictions, and will underestimate the overall cost of restrictions. When the level of restrictions imposed is small ($<$ 5% of total demand), this underestimate will be small.

3.6.3 Summary

In this section, the costs associated with water supply system failures have been considered. An overview of research undertaken in this area has been presented. Two approaches to the inclusion of water supply system restriction costs have been detailed. The first approach, where data concerning the price elasticity of demand is available, calculates the economic loss resulting from the imposition of outdoor water use restrictions, as the sum of the producer surplus loss and the consumer surplus loss. When data concerning the price elasticity of demand is not available, a simpler approach has been described which may be appropriate. This simpler approach does not consider the loss of consumer surplus. Where practical, the first approach is considered preferable.

3.7 Summary of Chapter

In this chapter, a simulation methodology for the assessment of reliability-cost tradeoffs for multiple reservoir headworks systems has been presented. In the methodology, a simulation/optimisation model is integrated with a number of models and procedures. These models and procedures are :

1. A synthetic inflow data generation model.
2. A synthetic demand data generation model.
3. A component reliability analysis procedure.
4. A procedure for assessment of economic costs associated with the imposition of water supply restrictions.

The methodology is applicable to the assessment of reliability-cost tradeoffs for any water supply headworks system. The influence on the reliability-cost performance of the four areas considered will vary according to the system configuration under consideration. In certain systems, some elements of the methodology may have only minor impact on the overall reliability-cost trade-offs for the system and need not be included.

Chapter 4

Application to the Adelaide Headworks System

4.1 Introduction

This chapter describes the application of the methodology presented in Chapter 3 to the metropolitan Adelaide water supply headworks system.

The chapter commences with a description of the metropolitan Adelaide water supply headworks system, and the rules that are used to operate it. A simulation/optimisation model, currently used by the Engineering and Water Supply Department of South Australia (EWS) to assist in the operation of the system is then detailed. This model has been used as the primary tool to test the application of the proposed methodology to the Adelaide system. Details of the synthetic inflow and demand data generation models developed for the Adelaide system are presented. The 'walking party' approach described in Section 3.5.1 of Chapter 3 for the assessment of component reliabilities has been applied to the bulk water transfer system within the headworks system. Details of the application of this approach are described. The frequency-duration analysis technique has then been applied to the data obtained from the 'walking party' approach and the results used as input to a Monte Carlo component failure generation model. Using this model a sequence of synthetic failure data has been generated. The chapter concludes with the application of an approach for the assessment of economic costs associated with the imposition of outdoor water use restrictions on the Adelaide headworks system.

4.2 Description of the Metropolitan Adelaide Water Supply and Storage System

This section describes the metropolitan Adelaide water supply and storage system and some of the issues associated with its operation. This system has been used as a case study for the methodology proposed in Chapter 3.

The metropolitan Adelaide water supply system caters for a population of approximately one million people and seeks to maintain a reliable supply of water at minimum cost. South Australia is the driest state in the driest continent in the world. Water is a precious commodity in South Australia and to maintain a water supply of suitable quality is a challenging task. Within the state, limited areas exist where sufficient runoff enable reservoirs to be economically constructed and operated. These locations are also of prime agricultural and horticultural value.

There are few remaining reservoir sites that would increase the available water supply to metropolitan Adelaide. At the same time, the available water supply from the existing reservoirs is insufficient in the majority of years to maintain supply to the metropolitan areas. This fact has long been recognised by the EWS and so a number of major pumping pipelines have been constructed to supplement the available catchment runoff with water from the River Murray. A schematic for the metropolitan headworks system is shown in Figure 4.1.

The quality of Adelaide's water has been the subject of criticism since the settlement's first public reticulation system commenced operation in 1860. The problems include those of turbidity, colour, taste and odour. The turbidity comes from very fine particles of clay, silt and algae suspended in the water. The colour is due to dissolved or colloidal material of vegetable origin. Taste and odour problems are mainly caused by algae and plankton. There are two main reasons why Adelaide's water suffers from these problems - poor natural catchments and an increasing reliance on the River Murray. Adelaide's natural catchment areas, the Mount Lofty Ranges, are small, closely settled and have a comparatively high rainfall in a largely arid state. Since the settlement of South Australia, the land in these catchment areas has been used for a range of intensive agriculture and this human occupancy and agricultural use has seriously affected the quality of water collected from these catchments. A similar situation applies to the River Murray, and some of its tributaries, particularly the Darling River, whose waters contain a high level of fine silts, particularly when in flood.

In 1967, the EWS commenced a detailed investigation into the characteristics of Adelaide's water supply which resulted in the publication of a report entitled 'Water Treatment for Metropolitan Adelaide' [88]. This report confirmed that the quality of Adelaide's water fell well short of acceptable standards. While the water had not resulted in public health problems, its bacteriological quality did not, despite continuous chlorination, meet the very high standards regarded as desirable. The report recommended that Adelaide's water supply should be filtered and this recommendation was adopted as a government policy. Since its publication, six water filtration plants have been constructed to meet the report's recommendations. Myponga water filtration plant was the last of these plants to be constructed and was opened in November 1993. All areas of metropolitan Adelaide are now supplied with filtered water. The six water filtration plants have been constructed in a modular fashion. Within each plant there are at least two of each basic operating unit, enabling normal maintenance and emergency repairs to be undertaken without the necessity for closure of the plant. During the period of maintenance or repair, the plant operates at reduced capacity and utilises balancing storages contained in the filtered water storage tanks downstream of the plant.

For the purpose of analysis and modelling, Adelaide's water supply and storage system can conveniently be divided into two subsystems :

1. The Southern System
2. The Northern System

These two systems have been considered as separate entities within the optimisation/simulation model and further discussion will relate to one or the other of these two systems.

4.2.1 The Southern System

The southern metropolitan water supply and storage system comprises three reservoirs and one pumping system from the River Murray. These are :

- Mount Bold Reservoir
- Happy Valley Reservoir
- Myponga Reservoir

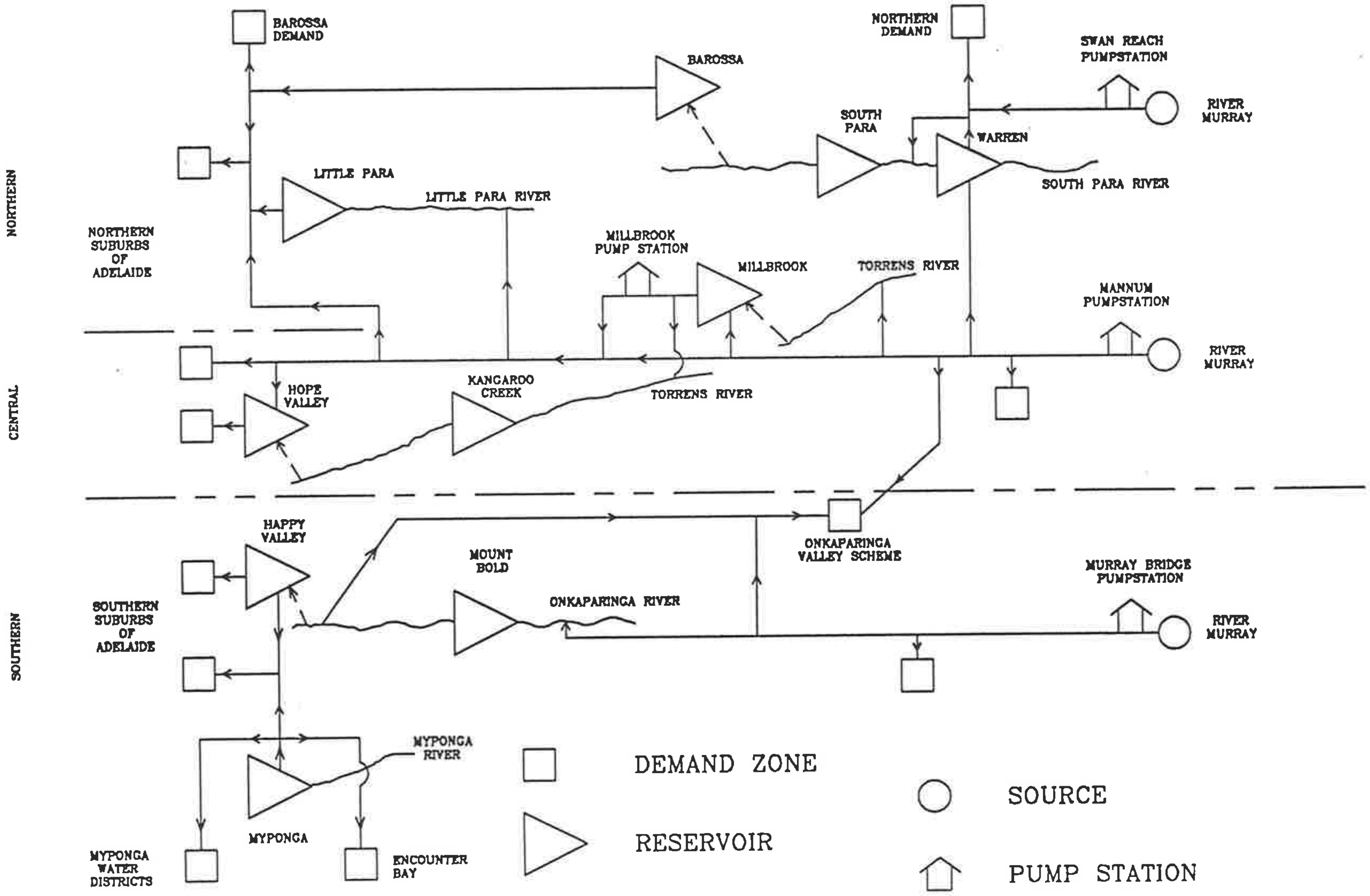


Figure 4.1: The Metropolitan Adelaide Water Storage System

- The Murray Bridge-Onkaparinga Pumping System

Two water filtration plants are used to provide a filtered water supply to the areas supplied by the southern system. These plants are :

- Happy Valley Water Filtration Plant
- Myponga Water Filtration Plant

A schematic showing the southern system as modelled is presented in Figure 4.2. The four demands on the southern system shown in this figure are described below :

PMBOO	:	Murray Bridge-Onkaparinga Pumping System On-line Demand
SDHV	:	Demand from Happy Valley Demand Zone
SDMY	:	Demand from Myponga Demand Zone
SDEB	:	Demand from Encounter Bay Demand Zone

Each of these major components will now be described in some detail.

Mount Bold Reservoir Construction of Mount Bold Reservoir began in 1932 and was completed in 1938. The reservoir is located on the Onkaparinga River approximately 10 km upstream from the township of Clarendon. Mount Bold Reservoir has a capacity of 45.9 GL and a catchment area of 388 km^2. Its storage can be supplemented with water pumped from the River Murray via the Murray Bridge-Onkaparinga pipeline. Water discharges from the pipeline into the Onkaparinga River near Verdun and flows down the river into Mount

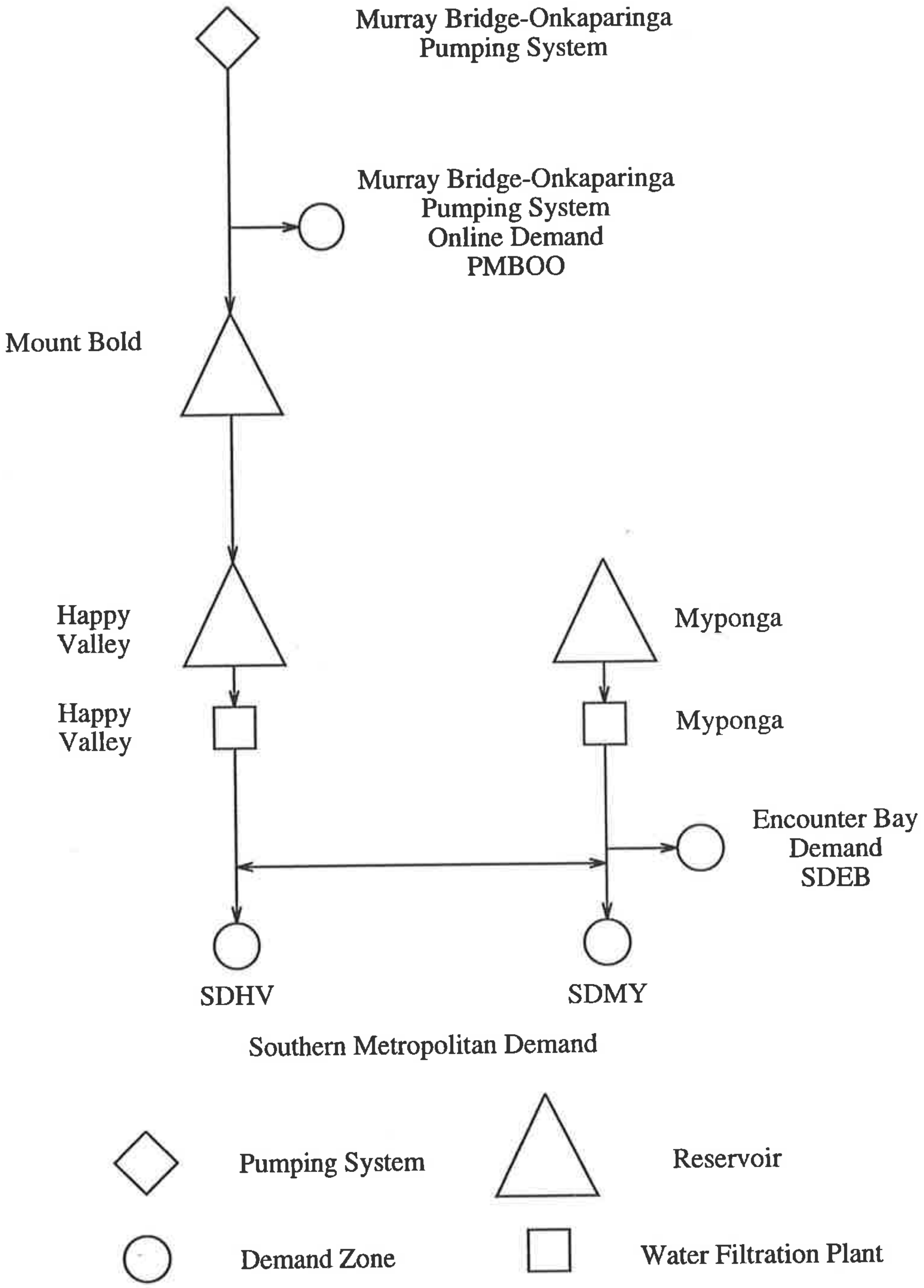


Figure 4.2: The Southern Metropolitan Water Storage System Schematic

Bold Reservoir. Mount Bold Reservoir is primarily used for storage and cannot directly supply metropolitan Adelaide. Water can be released from the reservoir and flows along the Onkaparinga River to Clarendon Weir. It can then be diverted at this weir to Happy Valley Reservoir via the Happy Valley diversion tunnel.

Happy Valley Reservoir Happy Valley Reservoir was constructed during the years 1892 to 1896 in response to increasing water demand due to the population growth in Adelaide. It is an off-stream supply reservoir and local catchment runoff is prevented from entering it by a catch drain surrounding the reservoir. Runoff from the catchment area downstream of Mount Bold Reservoir, but upstream of Clarendon Weir can be captured in Happy Valley Reservoir providing the capacity of the Happy Valley diversion tunnel is not exceeded. This catchment has an area of 57 km^2. The capacity of Happy Valley Reservoir has been surveyed as 12.7 GL.

Myponga Reservoir Construction of Myponga Reservoir commenced in 1957 and was completed in 1962. The reservoir is an on-stream storage and was constructed to supply the Myponga district. It is linked to the Happy Valley Reservoir via a trunk main and can supply a portion of the southern metropolitan demand. A pumping station adjacent to the Myponga trunk main near Sellicks Hill also allows Myponga Reservoir water to be used to supplement the Encounter Bay system. The Encounter Bay system includes the townships of Victor Harbor, Port Elliot, Middleton and Goolwa.

Myponga Reservoir has a catchment area of 124 km^2. Its storage is dependent on catchment runoff and <u>cannot</u> be supplemented with water pumped from the River Murray. Myponga Reservoir is both a storage and supply reservoir meeting the demands of the Myponga demand zone and the Encounter Bay system. The surveyed capacity of Myponga Reservoir is 26.8 GL.

The Murray Bridge-Onkaparinga Pipeline The Murray Bridge-Onkaparinga pipeline was commissioned in 1973 to transfer water from the River Murray to the southern metropolitan water supply system. It provides minor direct supplies to the Murray Bridge, Onkaparinga Valley and Nairne-Mount Barker distribution systems, however its main discharge point is into the Onkaparinga River near Hahndorf. There is considerable pumping flexibility in this pipeline and hence off-peak electricity tariffs can be used to advantage.

The pipeline is serviced by three pumping stations along its length. Each of these pump stations has three pump sets operating in parallel, having a total maximum combined pumping rate of 20.74 ML /hour. This corresponds to a maximum monthly rate of 14,900 ML. At the pumping station closest to the river at Murray Bridge, a fourth pump is installed to supply the Murray Bridge township. This small pump has not been included in the simulation/optimisation model of the Adelaide system. A longitudinal schematic section along the pipeline is shown in Figure 4.3. The on-line pipeline storages have only limited capacities and are primarily used during startup and shutdown of the system and to accommodate slight differences in the operational characteristics of the three pump stations.

The capacity of this pipeline is such that the weekly southern system demand could be met solely with pumped water. It must be noted however, that the peak daily demand exceeds the peak pumping capacity. Given a small storage, the daily and weekly demand loads could be met.

Further details of the individual pumping components making up the Murray Bridge Onkaparinga pipeline system are described in Section 4.6 with particular reference to their reliability characteristics.

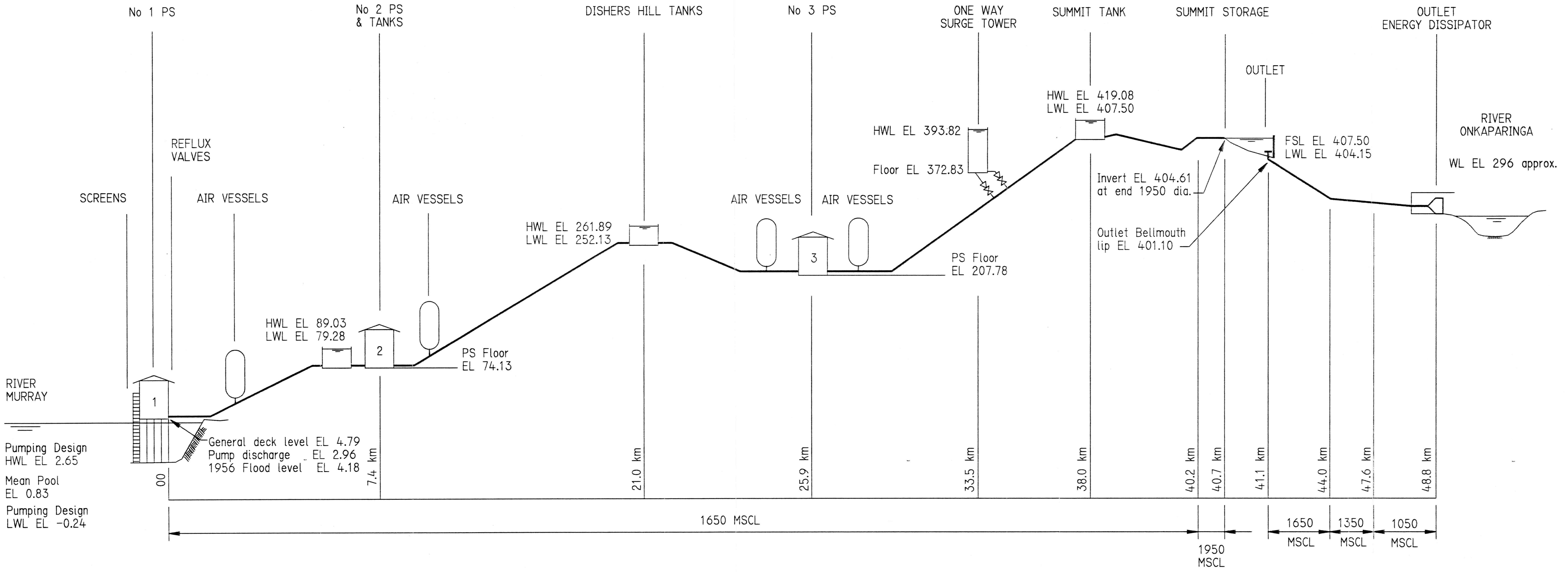


Figure 4.3: The Murray Bridge-Onkaparinga Pipeline Longitudinal Schematic Section

Happy Valley Water Filtration Plant Happy Valley water filtration plant was commissioned in two stages, the second stage being completed in November 1991. The first stage of the plant commissioned had a nominal process capacity of 500 ML/day and the second stage increased the capacity to 850 ML/day. The Happy Valley water filtration plant is the largest plant of its kind in Australia.

Myponga Water Filtration Plant Myponga water filtration plant is the last of the six water filtration plants supplying metropolitan Adelaide and was officially opened in November 1993. The first stage of the plant has a process capacity of 50 ML/day and the second stage of the plant, not due until the year 2005 depending on population growth, will bring the total plant capacity to 100 ML/day.

4.2.2 The Northern System

The northern metropolitan water supply and storage system comprises seven reservoirs and two pipelines from the River Murray. These can be subgrouped into the South Para subsystem, the Little Para subsystem and the Torrens subsystem.

Those in the South Para subsystem are :

- Warren Reservoir
- South Para Reservoir
- Barossa Reservoir
- The Swan Reach-Stockwell Pumping System

Those in the Little Para subsystem are :

- Little Para Reservoir

Those in the Torrens subsystem are :

- Millbrook Reservoir
- Kangaroo Creek Reservoir
- Hope Valley Reservoir

- The Mannum-Adelaide Pumping System

Four water filtration plants are used to provide a filtered water supply to the areas supplied by the northern system. These plants are :

- Barossa Water Filtration Plant
- Little Para Water Filtration Plant
- Hope Valley Water Filtration Plant
- Anstey Hill Water Filtration Plant

A schematic of the overall northern system as modelled is shown Figure 4.4. The six demands on the northern system shown in this figure are listed below :

PMAO	:	Mannum-Adelaide Pumping System On-line Demand
NDWR	:	Demand from Lower North Demand Zone

NDBR	:	Demand from Barossa Demand Zone
NDLP	:	Demand from Little Para Demand Zone
NDAH	:	Demand from Anstey Hill Demand Zone
NDHV	:	Demand from Hope Valley Demand Zone

The physical layouts of the respective subsystems of the northern system are shown in Figures 4.5 , 4.10 and 4.11.

Each of these major components will now be described in detail.

Warren Reservoir Warren Reservoir was constructed on the South Para River during the years 1914 to 1916 to provide on-stream storage to supply the Barossa Valley region and the lower northern areas of South Australia. It maintains this function today but can also contribute to the northern metropolitan Adelaide supply system through its connections with the South Para Reservoir.

Warren Reservoir has a catchment area of 119 km^2. Its storage can be supplemented with water pumped from the River Murray in two ways :

1. A branch main from the Mannum-Adelaide pipeline discharges water into the Warren Reservoir.
2. Water from the Swan Reach-Stockwell pipeline can be diverted southwards through the Warren trunk main into the reservoir.

The capacity of the Mannum-Adelaide branch main limits the volume of water that can be transferred through this pipe to 420 ML per month.

The capacity of the Swan Reach-Stockwell pipeline limits the volume of water that can be transferred from Swan Reach to Warren Reservoir to 2,020 ML

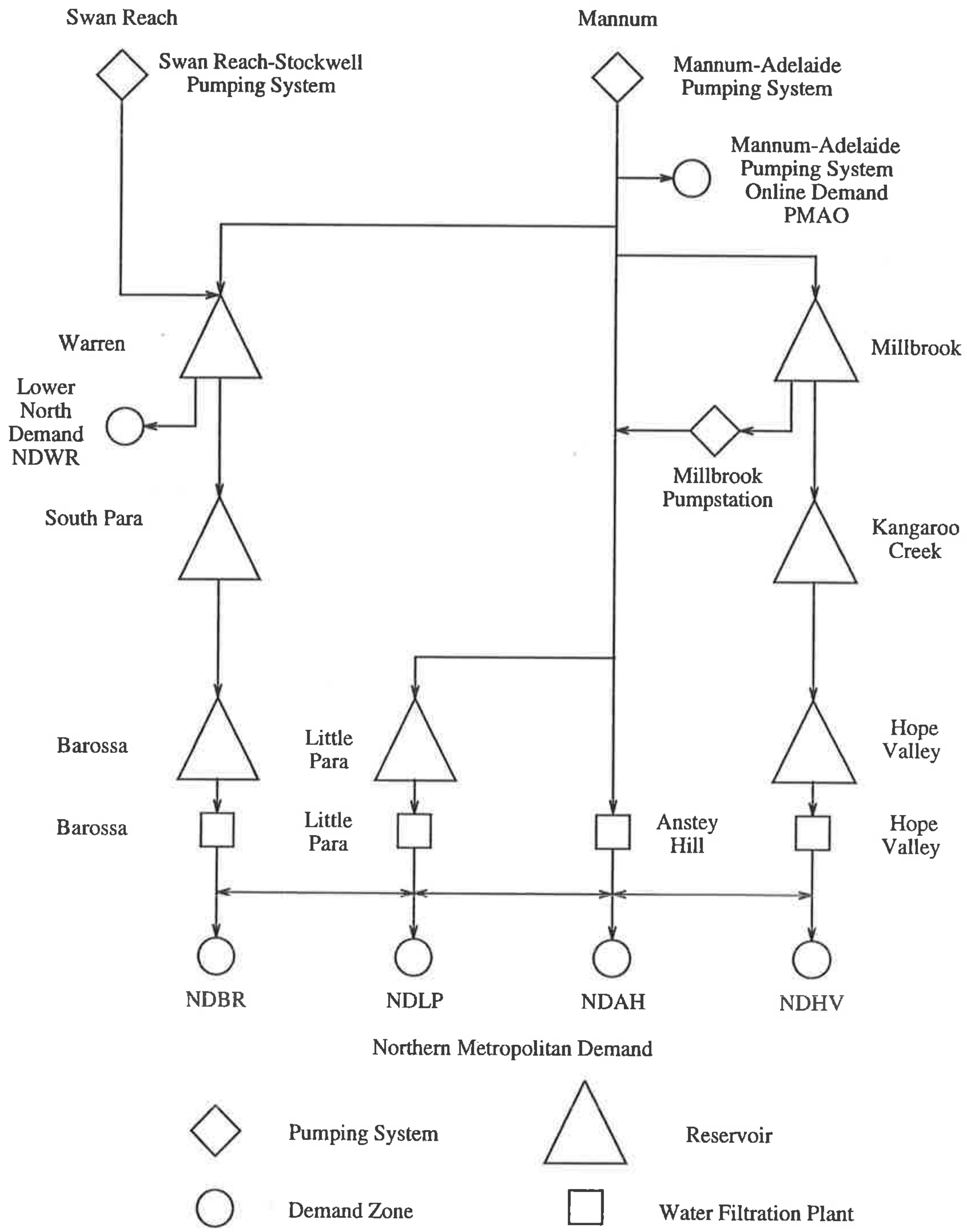


Figure 4.4: The Northern Metropolitan Water Storage System Schematic

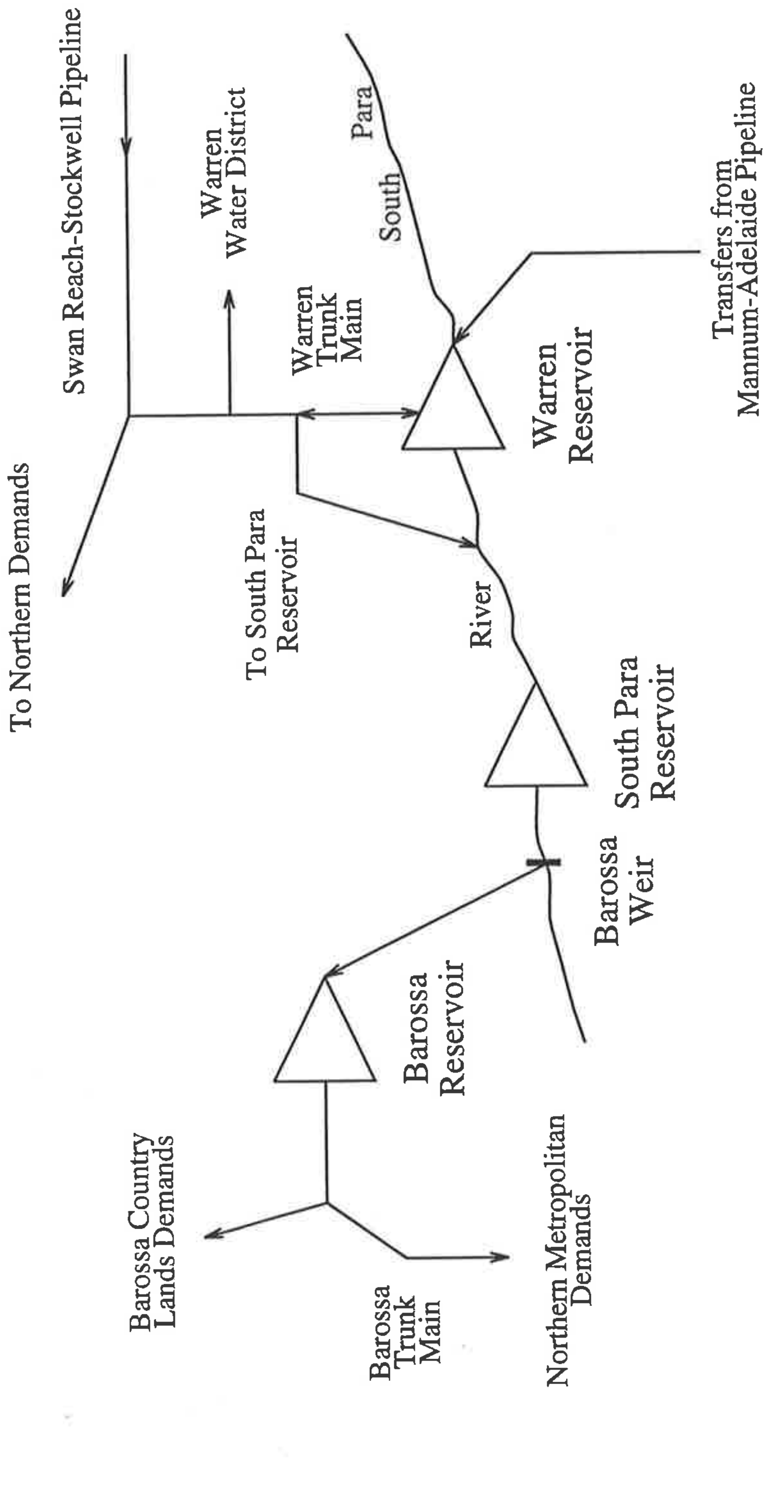


Figure 4.5: The South Para Water Storage Subsystem Layout

per month. The capacity of the Swan Reach-Stockwell pipeline to supplement the Warren Reservoir is further reduced by other demands for water from the pipeline. In 1982, Warren Reservoir was surveyed and found to have a capacity of 4.77 GL.

South Para Reservoir South Para Reservoir is the newest and largest of the three reservoirs in the South Para subsystem. It was constructed in anticipation of the rapid growth of residential and industrial areas between Adelaide and Gawler. Construction commenced in 1948 but due to the post-war demand on funds and resources, was not completed until 1958. South Para Reservoir is located on the South Para River at the confluence of Malcolm and Victoria Creeks.

It has a catchment area of 109 km^2. Its storage can be supplemented with water pumped from the River Murray via the Swan Reach-Stockwell pipeline diverted south through the Warren trunk main and released into the South Para River upstream of the reservoir. Water released or spilt from Warren Reservoir can also be used to supplement the reservoir storage.

In 1960, movable gates were installed on South Para Reservoir, which when closed could increase the reservoir storage capacity by 6,300 ML. In 1993, these gates were removed following concern regarding the stability of the gates under certain conditions. South Para Reservoir is the largest storage in the metropolitan Adelaide headworks system having a storage capacity of 44.8 GL.

Barossa Reservoir Barossa Reservoir was constructed between 1899 and 1902 as an off-stream storage. Local catchment runoff is diverted from entering the reservoir by a catch drain surrounding the reservoir. Water released or spilt from South Para Reservoir, together with runoff from the small catchment area downstream of South Para Reservoir, can be captured at Barossa Weir and transferred to Barossa Reservoir, providing the capacity of the Barossa diversion tunnel and aqueduct is not exceeded. The catchment, downstream of South Para Reservoir but upstream of Barossa Weir has an area of 7 km^2.

Initially, Barossa Reservoir was built to provide the Gawler township and surrounding areas to the west, south-west and north-west with a reliable water supply. Over the years the distribution system from the Barossa Reservoir has gradually extended to serve most of the area between Gawler and Port Wakefield. In 1940 the Barossa trunk main was built and after the second world war extended to supply the northern metropolitan satellite city of Elizabeth, linking Barossa Reservoir to the metropolitan supply system. As industrial and urban development proceeded and demand grew, a second trunk main was constructed and in 1976 a second outlet from Barossa Reservoir was built to supply this trunk main. The South Para and Barossa Reservoir storages now supply the northern metropolitan areas as far south as Salisbury as shown in Figure 4.6. The area of the demand zone supplied by these reservoirs varies from year to year and from summer to winter. The storage capacity of Barossa Reservoir has been surveyed as 4.51 GL.

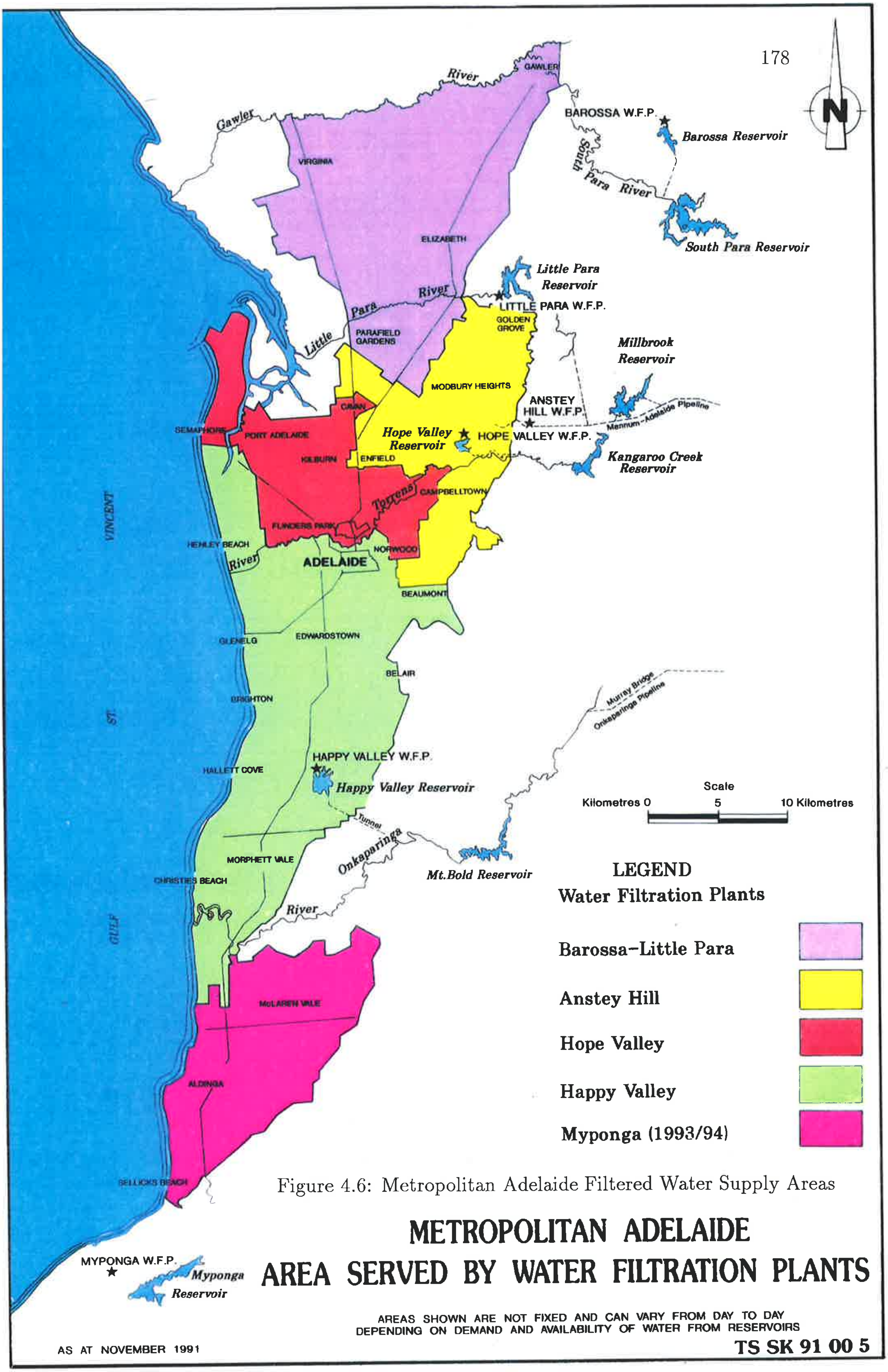


Figure 4.6: Metropolitan Adelaide Filtered Water Supply Areas

Swan Reach-Stockwell Pipeline The Swan Reach-Stockwell pipeline was commissioned in 1969 with the primary purpose of supplementing supplies to the lower north of the state, previously supplied solely from Warren Reservoir. Its secondary purpose was to supply townships and farmlands along its route. The pipeline utilises three pump stations and is 54 km in length. Each of the three pump stations contain three pump sets operating in parallel. The maximum effective pumping rate for the Swan Reach-Stockwell pumping system is 2,020 ML per month, making it the smallest of the three major pumping systems from the River Murray, supplying metropolitan Adelaide.

A longitudinal schematic section along the pipeline is shown in Figure 4.7. The on-line pipeline storages have only limited capacities and are primarily used during startup and shutdown of the system. They also are used to accommodate slight differences in the operational characteristics of the three pump stations.

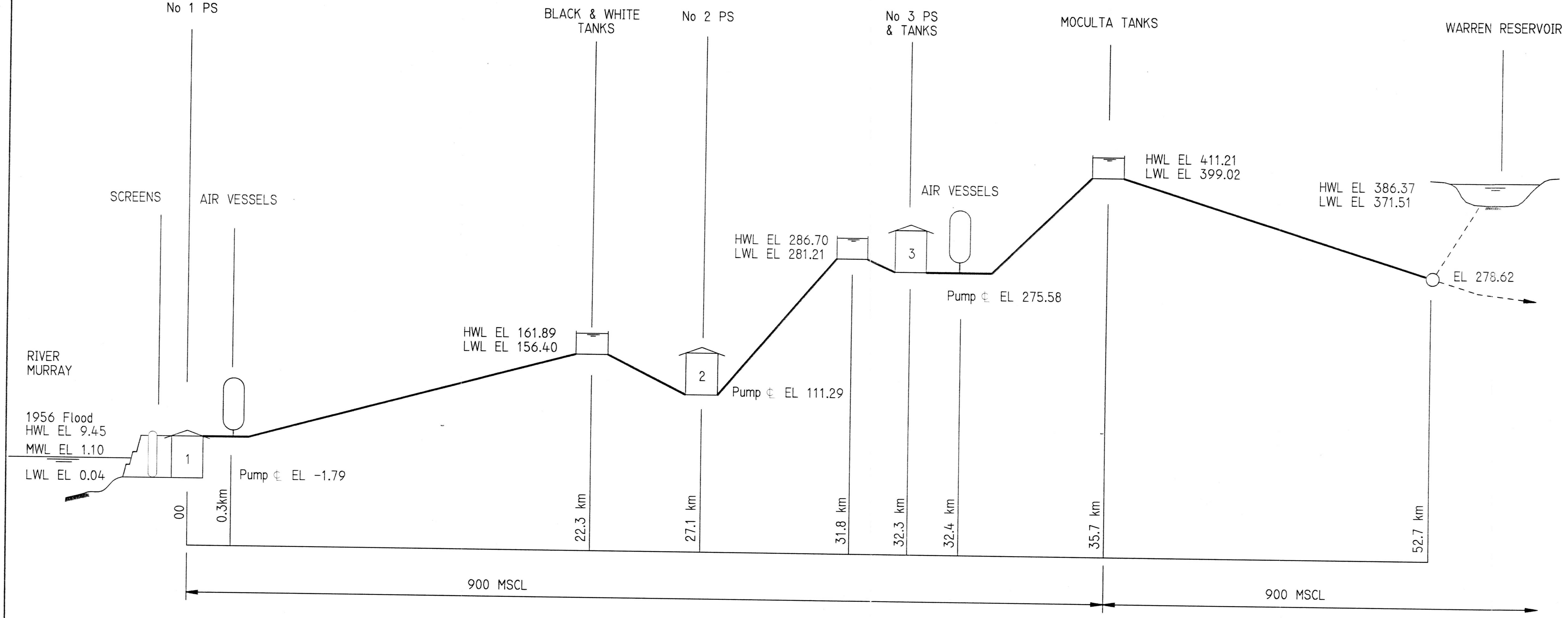


Figure 4.7: The Swan Reach-Stockwell Pipeline Longitudinal Schematic Section

Further details of the individual pumping components making up the Swan Reach-Stockwell pipeline system are described in Section 4.6 with particular reference to their reliability characteristics.

The Warren trunk main extends northwards from the Warren Reservoir and connects to the Swan Reach-Stockwell pipeline at Stockwell. From there it continues, gradually swinging westward to terminate at Paskeville at the top of the Yorke Peninsula. During periods of low demand on the Warren trunk main north of Stockwell, the Swan Reach-Stockwell pipeline can be used to supplement storages in both Warren and/or South Para Reservoirs by allowing water to flow southwards in the Warren trunk main. This water can be fed directly into Warren Reservoir or released into the South Para River just upstream of the South Para Reservoir.

Details of the pipeline alignment and capacity together with the formulation used within the northern system model are shown in Figures 4.8 and 4.9.

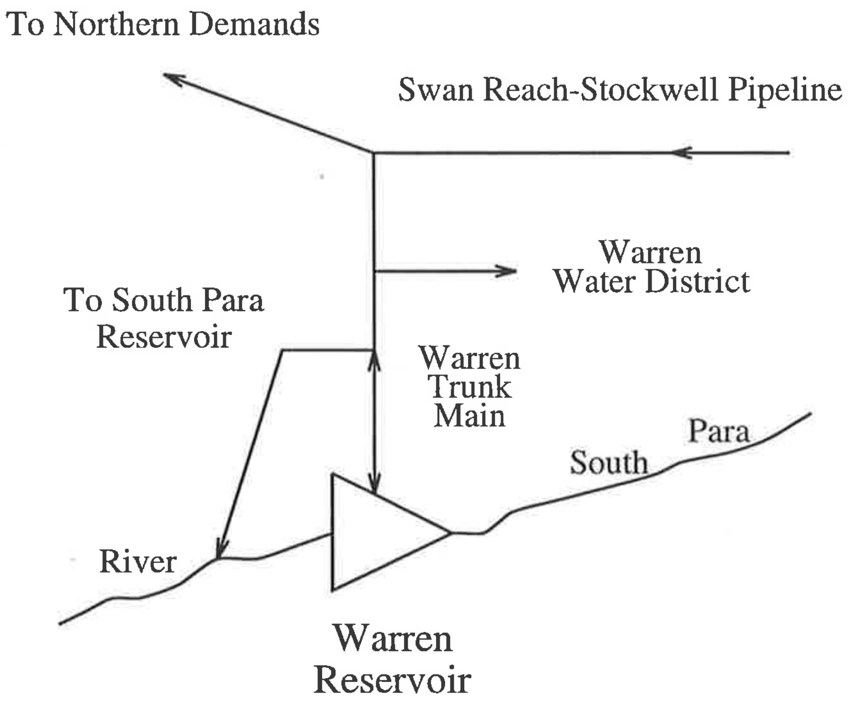


Figure 4.8: The Swan Reach-Stockwell Pipeline System Layout

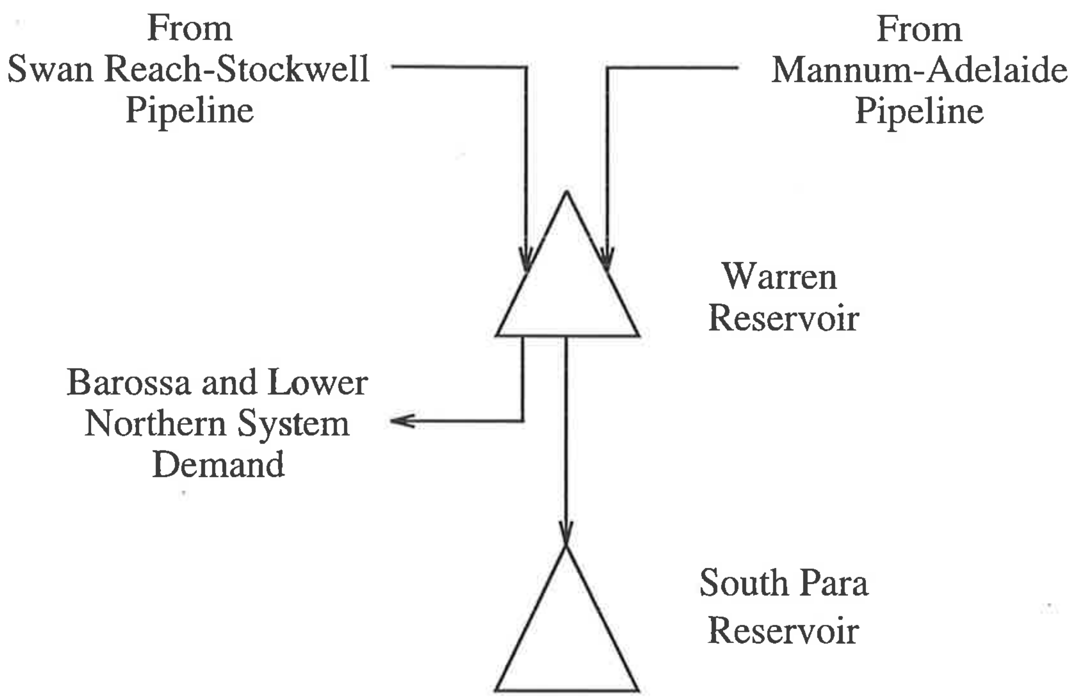


Figure 4.9: The Swan Reach-Stockwell Pipeline System Schematic

Little Para Reservoir Little Para Reservoir is the most recent addition to the metropolitan Adelaide reservoirs. It is constructed on the Little Para River and was commissioned in January 1979 having a storage capacity of 20.8 GL.

Little Para Reservoir has a catchment area of 83 km^2 with a mean annual yield of 7.5 GL. Its storage can be supplemented with water pumped from the River Murray via a branch main off the Mannum-Adelaide pipeline as shown in Figures 4.10 and 4.13. This branch main discharges into the Little Para River and the water then flows downstream into the reservoir.

Little Para Reservoir functions largely as a balancing storage for River Murray water, which can be pumped to the reservoir in winter when other demands on the Mannum-Adelaide pipeline are low. The reservoir is currently used to supply metropolitan Adelaide during summer months only, but with the rapid development of urban areas nearby it is likely to be used more extensively in the future. Releases from Little Para Reservoir are also used to supplement recharge of the groundwater table in the northern Adelaide plains.

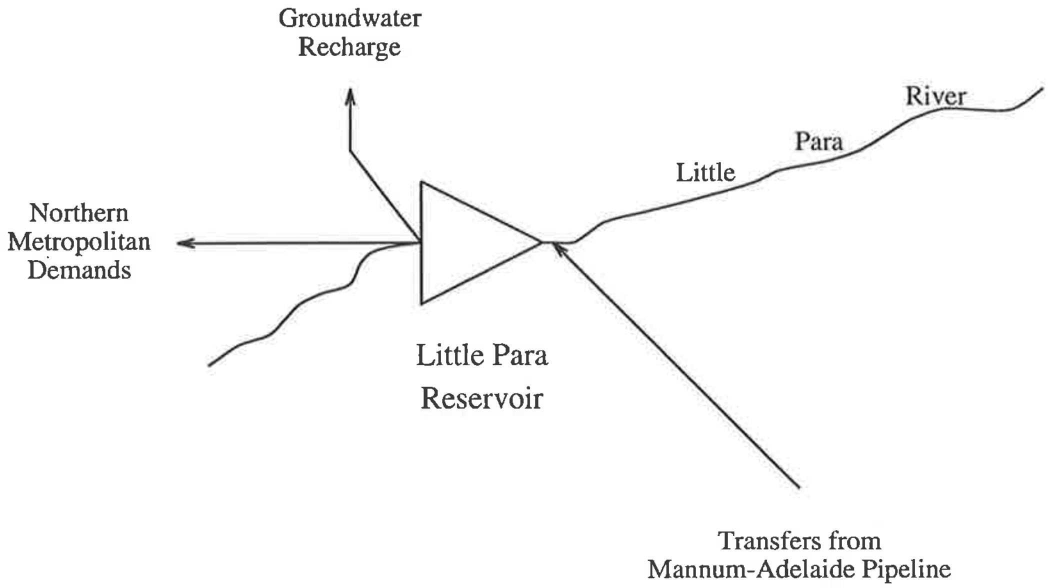


Figure 4.10: The Little Para Water Storage Subsystem Layout

Millbrook Reservoir Millbrook Reservoir was constructed during the years 1914-18 to control flows in the upper reaches of the River Torrens and to provide a reservoir with sufficient elevation to supply high level eastern suburbs by gravity. It was built as an off-stream reservoir on Millbrook Creek just below its confluence with Chain of Ponds Creek. These creeks supply a very small proportion of the intake water to the reservoir with its main source, the Torrens River, being diverted at Gumeracha Weir via the Millbrook diversion tunnel.

The catchment area for Millbrook Reservoir including the catchment area upstream of Gumeracha Weir is 233 km^2. Its storage can be supplemented with water pumped from the River Murray via the Mannum-Adelaide pipeline. Water can be released from the pipeline at a number of points and discharged into the Torrens River. The water then flows downstream to Gumeracha Weir and can be diverted into Millbrook Reservoir or allowed to continue down the Torrens River to Kangaroo Creek Reservoir.

Water can be released from Millbrook Reservoir into the Torrens River and hence transferred to Kangaroo Creek Reservoir. In 1971, an alternative outlet for Millbrook Reservoir was provided with the construction of the Millbrook pumping station. This pumping station lifts water from the reservoir into the gravity section of the Mannum-Adelaide pipeline. Millbrook Reservoir has a surveyed capacity of 16.5 GL.

Millbrook Pumping Station Millbrook pump station is located just downstream of the Millbrook Reservoir. The pump station was constructed to assist in meeting peak demands in the area served by the Mannum-Adelaide pipeline at Anstey Hill. The pump station provides considerable flexibility to the overall system and can be utilised to transfer water from the Torrens system into Little Para Reservoir as shown in Figures 4.11 and 4.13.

Millbrook pump station contains four pump sets operating in parallel and has a maximum capacity of 7,500 ML per month.

Further details of the individual pumping components making up the Millbrook pumping station are described in Section 4.6 with particular reference to their reliability characteristics.

Kangaroo Creek Reservoir Construction of Kangaroo Creek Reservoir was commenced in 1967 and was completed in 1969. The reservoir is situated 1 km downstream from the confluence of Torrens River and Kangaroo Creek and had a capacity on completion of 24.4 GL. In 1982-83, as part of the River Torrens Flood Mitigation Scheme (EWS [89]), modifications were carried out

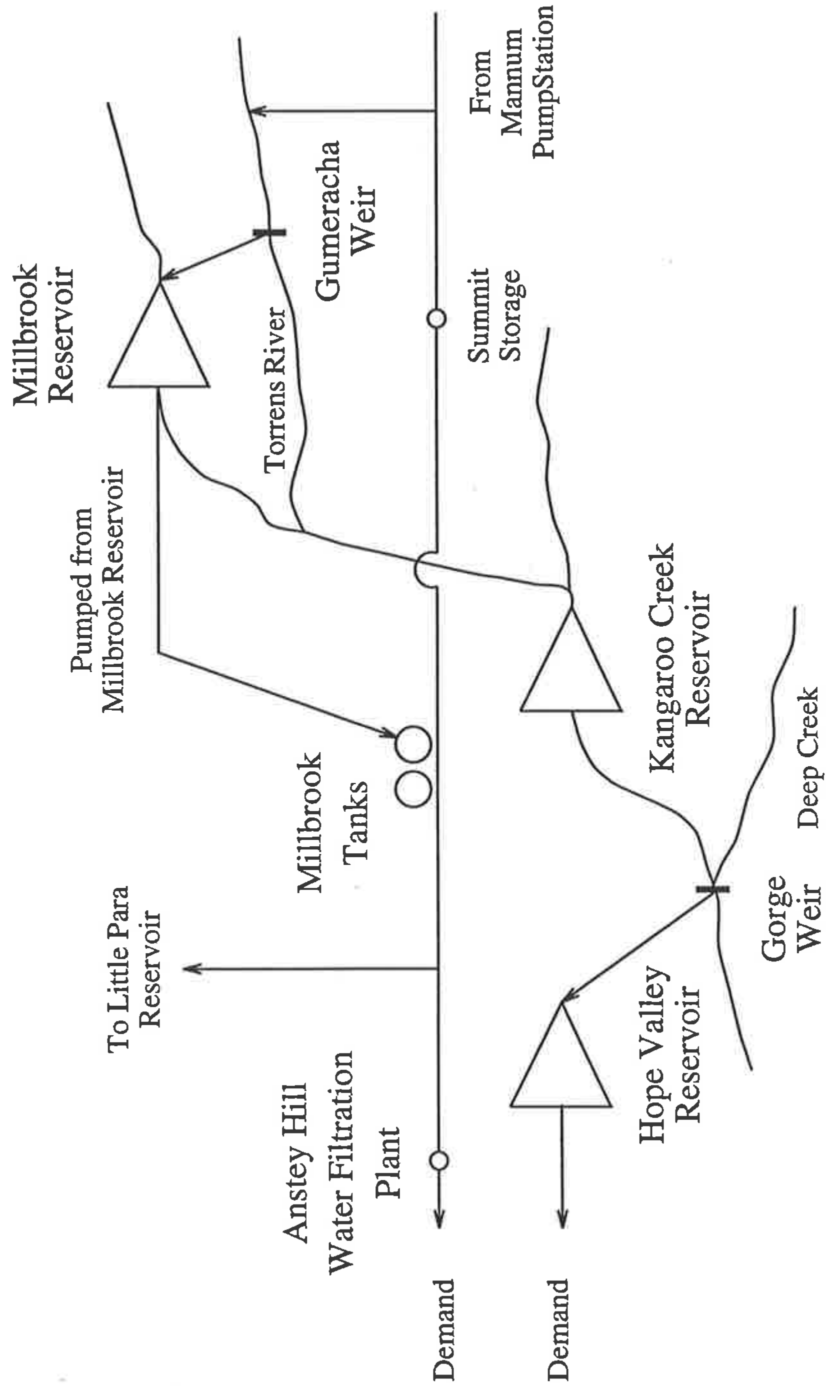


Figure 4.11: The Torrens Water Storage Subsystem Layout

on the dam, with the dam wall being raised by almost four metres and the spillway by three metres. Two 'escape' channels were cut in the spillway five metres below the level of the crest, effectively decreasing the capacity of the reservoir to 19.0 GL , but increasing its suitability for flood mitigation.

The catchment area supplying Kangaroo Creek Reservoir alone is 55 km^2. This area does not include the Millbrook Reservoir local catchment or the catchment area upstream of Gumeracha Weir.

Water can be released from Kangaroo Creek Reservoir into the Torrens River and hence transferred to Hope Valley Reservoir via the Gorge Weir and the Hope Valley diversion aqueduct as shown in Figure 4.11. The surveyed capacity of Kangaroo Creek Reservoir is 19.0 GL.

Hope Valley Reservoir Hope Valley Reservoir was the second of South Australia's water supply reservoirs and was constructed during the period 1869-71. The reservoir is an off-stream storage and local catchment water is prevented from entering the reservoir by a catch drain. Water released or spilt from Kangaroo Creek Reservoir, together with runoff from the catchment area downstream of Kangaroo Creek Reservoir, can be captured at Gorge Weir and transferred to Hope Valley Reservoir, providing the capacity of the diversion tunnel and aqueduct is not exceeded.

The catchment area for Hope Valley Reservoir alone, that is, the area downstream of Kangaroo Creek Reservoir but upstream of Gorge Weir is 57 km^2.

Hope Valley Reservoir is primarily used as a service reservoir from which water is distributed from the two other larger reservoirs in the Torrens subsystem. Hope Valley Reservoir has a surveyed capacity of 3.47 GL.

Mannum-Adelaide Pipeline The Mannum-Adelaide pumping system was the first means of supplying River Murray water to metropolitan Adelaide. The use of the River Murray as a source of water for Adelaide was considered for many years, but was not a practical option until the Goolwa Barrages at the river mouth were constructed preventing saline water entering the lower reaches of the river. The pipeline was commissioned in 1954, extending 60 km from the river township of Mannum to a terminal storage in the suburb of Modbury. There are three pump stations on the rising section of the pipeline and the pumping capacity of each pump station is 10,800 ML per month. A longitudinal schematic section along the pipeline is shown in Figure 4.12.

Figure 4.12: The Mannum-Adelaide Pipeline Longitudinal Schematic Section

Details of the individual pumping components making up the Mannum-Adelaide pumping system are described in Section 4.6 with particular reference to their reliability characteristics.

The capacity of the pipeline system is not uniform throughout its length and is dependent on the pipeline size. Details of the pipeline alignment and capacity are shown in Figures 4.13 and 4.14.

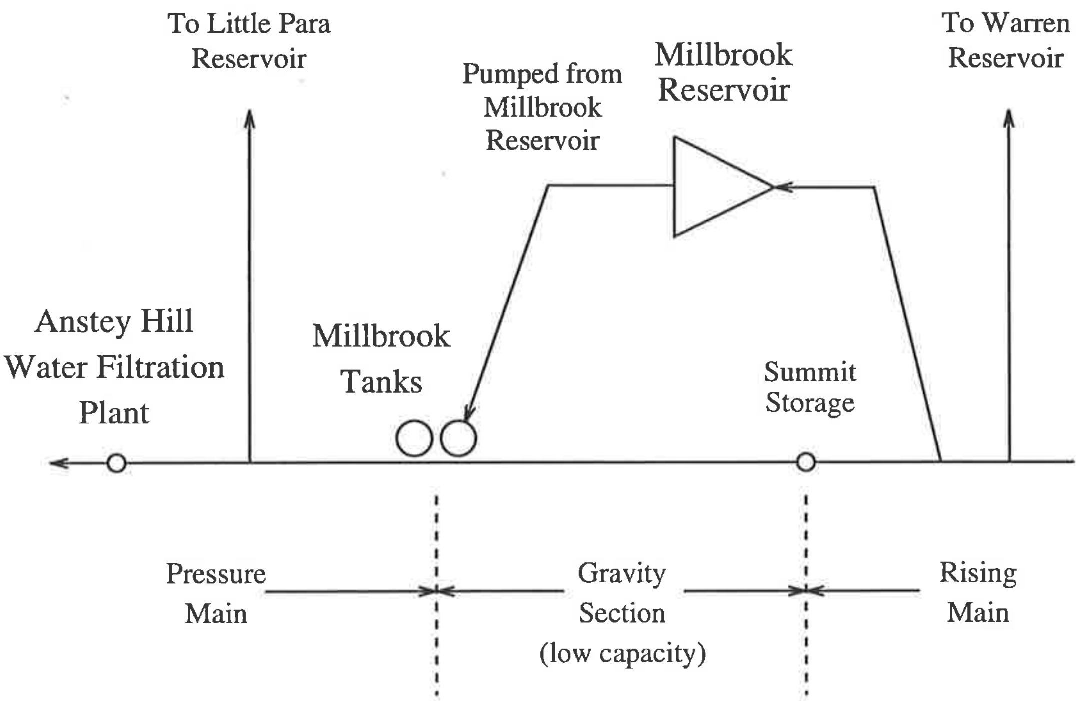
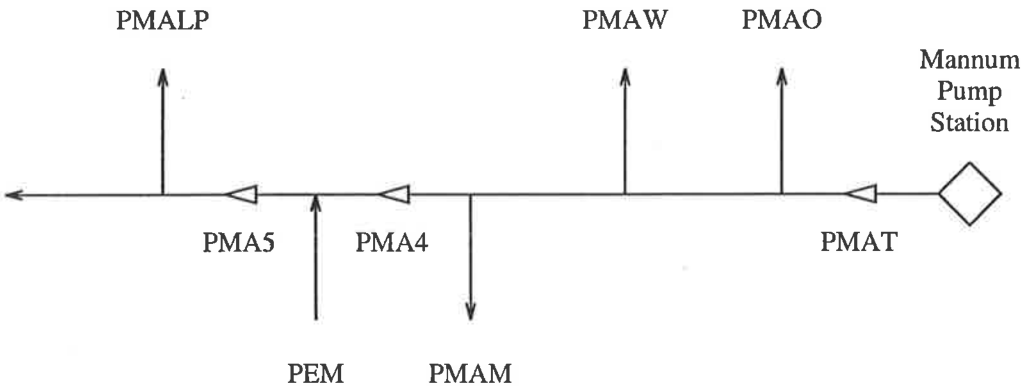


Figure 4.13: The Mannum-Adelaide Pipeline System Layout

Within the model used to simulate the operation of the system, the following parameter names have been adopted. These parameters have been used on the pipeline system schematic shown in Figure 4.14.

PMAT	:	Mannum-Adelaide Pumping System Total Capacity
PMAO	:	Mannum-Adelaide Pumping System On-line Demand
PMAW	:	Mannum-Adelaide to Warren Transfer Capacity
PMAM	:	Mannum-Adelaide to Millbrook Transfer Capacity

PMA4	:	Mannum-Adelaide Capacity between Summit Storage and Millbrook Tanks
PEM	:	Ex-Millbrook Pumping Capacity
PMA5	:	Mannum-Adelaide Capacity between Millbrook Tanks and Anstey Hill Water Filtration Plant
PMALP	:	Mannum-Adelaide to Little Para Transfer Capacity



Flow Constraints

PMAT <= 10.8 Gl/Month
PMA4 <= 6.6 Gl/Month
PMA5 <= 9.0 Gl/Month
PMAW <= 0.42 Gl/Month
PEM <= 7.5 Gl/Month
PMALP <= 5.4 Gl/Month

Figure 4.14: The Mannum-Adelaide Pipeline System Schematic

Barossa Water Filtration Plant Barossa water filtration plant was the third of six plants to supply Adelaide with filtered water and was opened in October 1982. The nominal capacity of the plant is 160 ML /day, which matches the hydraulic capacity of the Barossa trunk main.

Little Para Water Filtration Plant Little Para water filtration plant is located just downstream of the Little Para Reservoir. It was the fourth of the six water filtration plants to supply metropolitan Adelaide and was opened in November 1984. The nominal process capacity of the plant is 160 ML /day.

Hope Valley Water Filtration Plant Hope Valley water filtration plant was the first water filtration plant constructed in South Australia to supply the metropolitan Adelaide with filtered water. The plant was commissioned in September 1977 and has a design capacity of 273 ML/day.

Anstey Hill Water Filtration Plant Anstey Hill water filtration plant was the second of the six water filtration plants to supply metropolitan Adelaide. The plant became fully operational in January 1980 and has a total capacity (including overload) of 344 ML /day.

4.2.3 The River Murray

As previously described in this section, the Adelaide headworks system is dependent on the River Murray as a source of supply. Two issues associated with this supply source warrant specific consideration. These are water quantity and water quality.

4.2.3.1 River Murray Water Quantity Issues

The River Murray is the major water resource in Australia and forms part of the Murray-Darling drainage basin. This basin has a catchment area of one million square kilometres, which comprises nearly one seventh of the area of Australia as shown in Figure 4.15.

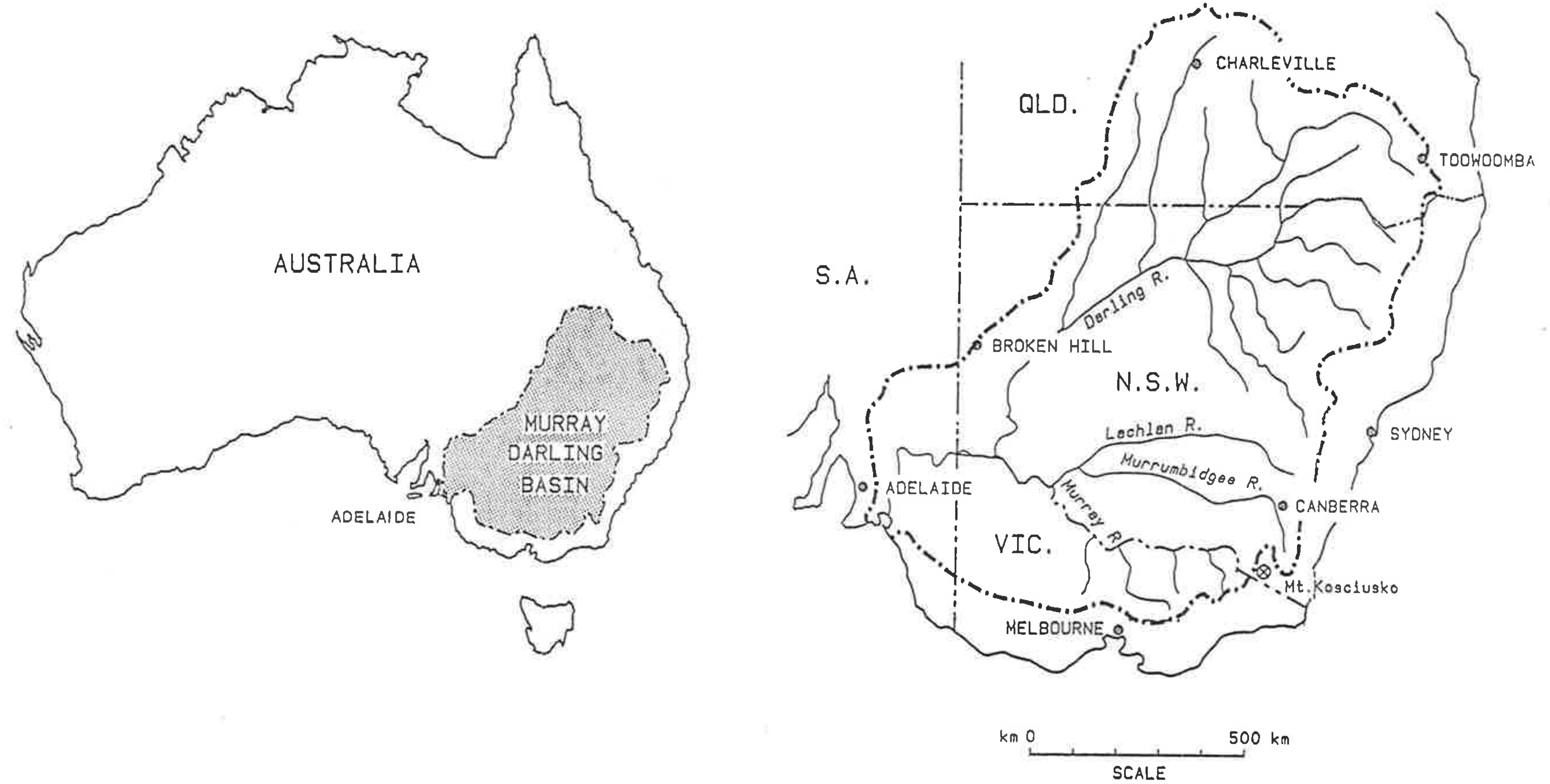


Figure 4.15: Adelaide and the Murray-Darling Basin

Storages within the Murray-Darling basin and an interstate 'entitlement' agreement ensure that River Murray water is readily available for Adelaide's water supply even under conditions of extreme drought. The location of the city of Adelaide and the major pipelines in relation to the River Murray is shown in Figure 4.16.

Entitlement flows to South Australia are measured at the border of South Australia with Victoria and New South Wales. These flows are used to supply water for a range of uses including irrigation, private stock and industrial water supply, domestic water supply, salinity dilution and storage evaporation losses from Lake Alexandrina and Lake Albert.

Private stock, industrial and domestic water supply, excluding the quantities pumped to the metropolitan Adelaide, have been relatively constant over the last ten years. The highest level of water use over this period is shown in Table 4.1 and occurred during the 1988/89 water year.

Month	1988/89 Water Use (GL)	Assumed Maximum Water Use (GL)
July	1.74	2
August	2.76	3
September	3.05	4
October	4.47	5
November	4.53	5
December	4.78	6
January	6.54	8
February	5.96	6
March	4.56	5
April	3.22	4
May	2.32	3
June	1.75	2
Annual	45.7	53

Table 4.1: River Murray Stock, Industrial and Domestic Water Use in South Australia excluding Pumping to Metropolitan Adelaide

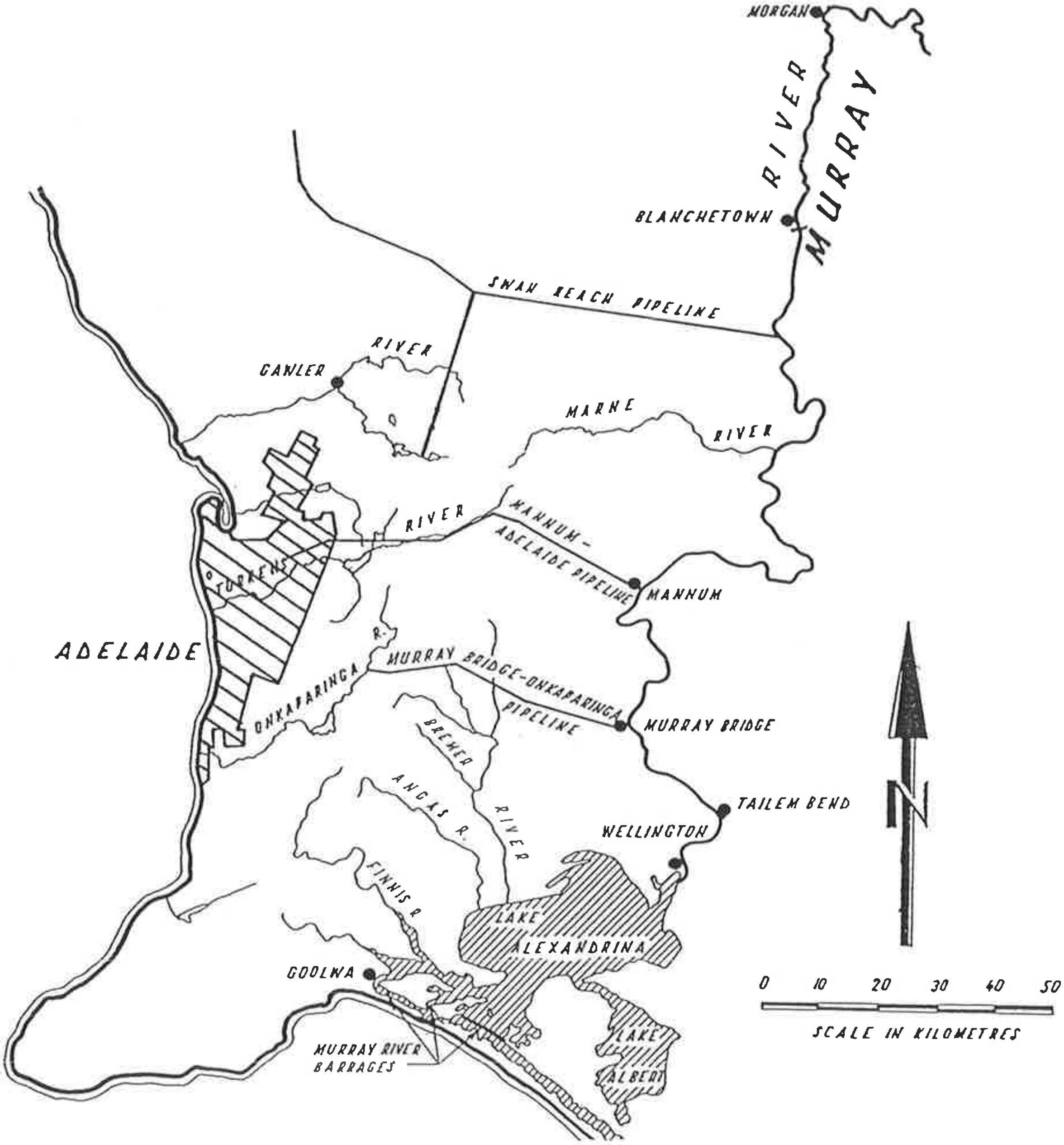


Figure 4.16: Pipelines from the River Murray to the Adelaide Reservoirs

Month	Entitlement Flow at the South Australian Border (GL)	Irrigation and Lake Evaporation Allocations During Entitlement Flow Years (GL)	Assumed Maximum Other Water Use (GL)	Entitlement Flow Available for Pumping to Metropolitan Adelaide (GL)
July	109	53	2	54
August	124	82	3	39
September	135	100	4	31
October	170	118	5	47
November	180	145	5	30
December	217	179	6	32
January	217	179	8	30
February	194	147	6	41
March	186	125	5	56
April	135	95	4	36
May	93	60	3	30
June	90	54	2	34
Annual	1850	1337	53	460

Table 4.2: River Murray Entitlement Flows within South Australia

The minimum volume of water available for pumping to the metropolitan Adelaide during an 'entitlement' year, while providing for irrigation allocations, evaporation loss allocations for the lower lakes (Lake Alexandrina and Lake Albert) and other maximum water uses, is presented in Table 4.2.

The maximum pumping rate possible to the metropolitan Adelaide headworks system from the River Murray through the three major pumping systems is 27.5 GL per month.

Water licences have been allocated to irrigators along the river in South Australia. When monthly entitlement flows to South Australian cannot be supplied, irrigation allocations are reduced by the ratio of the actual supplied flows to the entitlement flows. Seven in ten years, South Australia receives in excess of its entitlement flows. Close [41] estimated that the recurrence interval for South Australia to receive approximately 75% of its entitlement is 100 years, based on the 100 years of historical data modified for the current configuration and operating rules for the Murray-Darling basin system. Work has been undertaken by Burton [25] examining the performance of the Murray-Darling basin under extreme events for the current system configuration and operating rules. Burton generated 84 replicate sets containing 95 years of monthly data for the system. Analysis of this generated data under the current operating rules for the system showed that if the total active storage in the system on the 31st of July in a given year was less than 1000 GL , the probability of South Australia receiving more than 70% of its entitlement flows for the following two years was 92%, increasing to 100% in the third year. If the proportion was lowered to 65% of South Australia's entitlement flow, the probability increased to 100% for all following years. Even in the event of extremely low flows occurring in the River Murray into South Australia, political priorities placed on the use of the River Murray water would ensure pumping requirements for metropolitan Adelaide were adequately met, over and above irrigation requirements.

If flow in the River Murray were reduced to zero at the intake points to the pump stations to metropolitan Adelaide, it would still be possible to pump water from the river to the metropolitan headworks system. By extending the intake lines into deeper sections of the river, a portion of the storages in Lake Alexandrina and Lake Albert would be accessible at Swan Reach, Mannum and Murray Bridge.

The likelihood of sufficient quantity of water being unavailable in the River Murray system to meet metropolitan Adelaide requirements is considered negligible. It has therefore been assumed that quantity requirements for pumping from the River Murray to the metropolitan headworks system are not limited by the availability of sufficient quantities of water in the River Murray.

4.2.3.2 River Murray Water Quality Issues

From time to time algal blooms occur in the metropolitan Adelaide reservoirs. These outbreaks can be appropriately managed to prevent the exposure of consumers to water supply health risks. When the presence of algae within a reservoir exceed certain levels, the reservoir is treated with an algicide (copper sulphate) which prevents the formation of an algal bloom. While this approach is entirely effective in enclosed bodies of water such as reservoirs, it is not effective in watercourses such as the River Murray.

Water quality in the River Murray has been an issue since European settlement of South Australia. The nature and climatic conditions of the Murray-Darling basin are such that the possibility of algal outbreaks occurring in the river will continue in the foreseeable future.

The critical months for algal blooms in the River Murray are January, February and March, with February being the key month. These months are during the middle of summer when temperature and flow conditions in the river system are ideal for algal growth.

Blooms occurring within the River Murray can potentially last up to 4 weeks and occur with 1 to 2 weeks warning.

Algal levels within the River Murray can rise markedly due to two processes :

1. Localised blooms that have occurred in backwaters during high river flows are flushed back into the river on the recession of the flood event. These events tend to be short lived and can be prevented in part by the use of floating booms across the backwater entrance.

2. Widespread algal blooms along the length of the river caused by low flow and warm conditions within the river system.

Only the occurrence of the second type of algal blooms are likely to impact significantly on the long term operation of pumping from the river.

Contingency plans have been developed to ensure that all consumers taking water from the metropolitan Adelaide headworks system receive a supply that is safe to drink. These contingency plans include such measures as :

- Use of activated carbon in water filtration plants to reduce toxins.
- Management of flows in the River Murray to move blooms away from pump intakes.
- Installation of floating booms around pump intakes to prevent the entry of floating algal scums.
- Changes of supply sources where possible.

The EWS has adopted a policy that water supplies will <u>not</u> be shut down in the event of a toxic bloom as this action would merely create other risks such as reduced firefighting capacity and reduced sanitation. Naturally, if contaminated water has to be supplied this would be heavily publicised.

Work is currently being undertaken to examine possible techniques for reducing the length and severity of the later type of algal blooms in the lower reaches of the River Murray. By inducing rapid, short period flow variations through operation of the river locks and weirs, it may be possible to break up and disperse algal blooms within the river.

The pumping of River Murray water containing high algal levels is inadvisable, however the following considerations do need to highlighted. For the three major pumping systems supplying metropolitan Adelaide, the current configuration of the headworks system enables all pumped water to be transferred to a storage reservoir before being treated in a water filtration plant. For the Murray Bridge-Onkaparinga pumping system, pumped water is discharged into Mount Bold Reservoir before being transferred to Happy Valley Reservoir. For the Mannum-Adelaide pumping system, pumped water can be transferred to Millbrook Reservoir and relifted as required at the Millbrook pump station. For the Swan Reach-Stockwell pumping system, pumped water can be discharged into Warren Reservoir. The transfer and discharge into these reservoir storages can be used to dilute, break down and kill the algae. Toxins and taste and odour problems can be treated using granular activated carbon (GAC) and other facilities within the water filtration plants.

Water quality issues associated with the River Murray are very important to the supply of water to metropolitan Adelaide. Problems with the turbidity and colour of source waters can be accommodated using the water filtration plants constructed to filter the metropolitan Adelaide water supply. Salinity issues are more difficult to remedy. Previous work by Crawley and Dandy [46] [47] [62] [65] has examined the inclusion of salt damage costs on the optimum operation of the metropolitan Adelaide water supply system. The inclusion of these costs results in significant changes to the optimum operating strategies for the system. The salt damage costs have not been included in this thesis as the principle focus is on the water quantity issues.

Within this examination of reliability-cost tradeoffs for the Adelaide, it has been assumed that the quantity requirements for pumping from the River Murray to the metropolitan headworks system are not limited by the quality of water available from the River Murray in terms of algal content, turbidity, colour or salinity.

4.2.4 Historical Operation of the Adelaide Headworks System

The operation of the Adelaide metropolitan water supply and storage system has historically been dependent on the experience of pumping engineers within the EWS. In order to manage the system competently, these engineers have used their experience together with available records of historical rainfall, streamflow and system demand to formulate a 'pumping programme' that meets the operational policy objectives of the department.

The current operational policy objectives are :

1. No operationally avoidable restrictions in the supply to consumers shall be imposed.
2. Costs shall be minimised subject to (1) above.
3. Improvements in the quality of water supplied shall be achieved whenever possible without significantly changing total costs.

The formulated 'pumping programme' is based on deterministic forecasts of future inflows and demands used to estimate the monthly variations in reservoir storages over the whole of the water year, July to June. The resulting reservoir storages provide the pumping engineers with the required pumping from the River Murray to maintain adequate supply levels and safe storages in the reservoirs. These levels are referred to as 'target storage levels' and are described in greater detail in Section 4.2.4.1.

This predicted strategy is updated each month (and sometimes at shorter time intervals) as variation between forecast and actual inflow and demand volumes occur. As the water year progresses, the time window considered by the 'pumping programme' gradually shrinks until only the final month of the water year is considered. At the start of the new water year a twelve month period is once again considered.

4.2.4.1 Reservoir Target Storage Levels

The 'target storage levels' comprise the following two components :

- Nominal Minimum Operating Level Component

- Demand Storage Component

These are determined in the manner described below for each month of the year for each reservoir in the headworks system.

Because Myponga Reservoir cannot be supplemented with water from the River Murray, an 'end of year' target storage is used. This target has been set to ensure supply of two consecutive years of local demand and 90% exceedance intakes. This 'end of year' target is used as the summer and winter nominal minimum operating level component of the reservoir target storage levels.

In addition to the nominal minimum operating level component, a demand storage component is added to obtain the target storage levels. This demand storage component comprises the volume of water necessary to meet the average demand for a specified period into the future from the month under consideration.

A schematic showing a generalised reservoir cross-section is given in Figure 4.17.

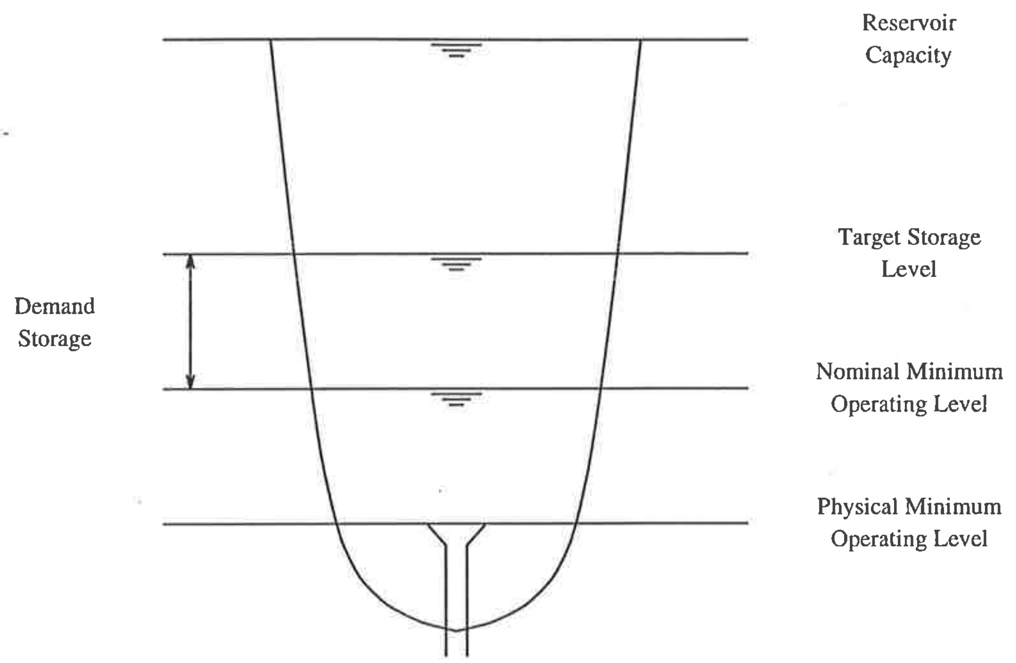


Figure 4.17: Reservoir Storage Definitions

4.2.4.2 Forecast Reservoir Inflows and Demands

As part of the operating rules for the Adelaide water supply system, a set of monthly forecast reservoir inflows and system demands are assumed at the beginning of each water year. On the basis of these forecast inflow and demand data sets a pumping program is prepared that will satisfy the reservoir target storage levels and minimise the operating costs for the system subject to the pumping, transfer and storage capacity constraints in the system. As the water year progresses, these forecast inflow and demand values are updated with actual inflow and demand figures and the pumping program amended as required.

Using the historical records of reservoir inflows a set of monthly inflow exceedance volumes have been determined. These inflow exceedance data sets are presented in Appendix B for the inflow exceedance probabilities ranging from 90% to 10%.

Historically, the 90% inflow exceedance data set and the 1982/83 water year demand volumes have been assumed for the preparation of the pumping program.

4.2.5 Current Operation of the Adelaide Headworks System

The target storage levels used in the current operating procedures for the Adelaide headworks system are given in Appendix B comprising the nominal minimum operating level component in Tables B.4 and B.14, and the demand storage component in Tables B.5, B.15 and B.16.

These latter levels correspond to one months demand storage for all reservoirs except Myponga, where the 'two year' criterion has been applied. These operating rules were introduced during the 1993/94 water year in response to preliminary results obtained from the research outlined in this thesis. Prior to these changes, the target storage levels for all reservoirs except Myponga corresponded to two months demand storage during most of the year and one months demand storage during April, May and June.

The physical minimum operating levels for each of the reservoirs are given in Chapter 5 in Tables 5.1 and 5.17. A comparison of these values with the nominal minimum operating level components of the target storage levels, reveals that for certain reservoirs the adopted nominal minimum operating level component is considerably higher than the physical minimum operating level of the reservoir. In reality, this means that the quantity of demand storage available in the event of failure of a component of the system would be greater than suggested by the particular operating rule description (eg. 4 weeks demand).

4.2.6 Summary

In this section, the metropolitan Adelaide water supply headworks system has been described. The necessity for catchment runoff water to be supplemented with water pumped from the River Murray has been highlighted. For the purpose of analysis and modelling, the overall headworks system can be divided into the southern and northern components. System layouts and schematics have been presented for various reservoir and pipeline components of these two systems. Where possible, these systems have been simplified to assist in the modelling process. The reliability of the River Murray as a supply source for metropolitan Adelaide has been examined in terms of water quantity and quality. Finally the historical and current operating rules for the metropolitan Adelaide headworks system have been described.

4.3 Description of the Headworks Optimisation Model - Adelaide (HOMA)

Over the last 7 years, work has been undertaken at the University of Adelaide in the Department of Civil and Environmental Engineering concerning the optimisation of the operation of multiple reservoir headworks system. During this period, an optimisation model has been developed for the metropolitan Adelaide headworks system and implemented on EWS computers. This model is entitled the Headworks Optimisation Model - Adelaide (HOMA) and it is currently used by the pumping engineer at the EWS to assist in the planning and operation of the headworks system. This model uses a set of target storages and inflow and demand forecasts as specified by the pumping engineer and determines the optimum operating strategy for these operating rules. Details of the early development of this model have been described by Crawley and Dandy [50] [51]. Further enhancements of the model have occurred as part of this thesis and a complete description of the current model is therefore included.

4.3.1 Overview of HOMA

HOMA is a suite of computer models, designed and written specifically to assist in the planning and operation of the metropolitan Adelaide headworks system.

The model has been developed to run on UNIX based machines and has been written to be as portable as possible between platforms. Hardware and software requirements for the model are :

- UNIX operating system.
- Ansi 'C' compiler.
- Fortran-77 compiler.
- 'X' Window system environment.

The HOMA model is a complex suite of programs originally written to operate on both SUN-3 and SUN-4 machines under the SunOS (Unix) operating system. It comprises computer code written in :

- 'C'
- 'ShellScript'

- 'Fortran'

and comprises a proprietary linear programming solution package entitled LINDO developed by LINDO Systems Inc.. Within the 'C' code , extensive use of 'X' routines have been made primarily using the Athena Widget set, although with some reference to the HP Widget set. HOMA has been developed with user friendliness in mind and utilises a mouse and graphics to interface between the user and the computer on which the model is run.

HOMA uses two separate components to model the southern and northern systems of the metropolitan Adelaide water supply system. During each model run, a working directory is created with a user specified name that contains the files required and produced by the model run.

HOMA uses a monthly time step and considers a fixed planning window of one water-year commencing at the beginning of July and ending at the end of the following June. As the year progresses, the planning window reduces in size until the next water year commences. In this way, HOMA emulates the decision making process required of the pumping engineer in the planning and operation of the headworks system.

The model can be applied in three different modes :

- Operational

  The operational mode of HOMA is of primary use in assisting the operations engineer in determining the pumping program for the current water year. It involves a single optimisation run to determine the optimum operating strategy up to the end of the current water year.

- Planning - Forecast

The planning - forecast mode of HOMA is of use in assisting planners, operators and the operations engineer in the examination of changes to the metropolitan headworks system and the impact of these changes on the operating costs for the system. In this mode, HOMA is only given the information that would have been available to the pumping engineer in the actual operation of the system.

An optimum operating policy is obtained for the system at a particular point in time. This policy for the coming month is then assumed to have been adopted. The forecast inflows and demands are then replaced with actual figures and a new formulation of the linear programming model constructed.

Where significant differences exist between 'forecast' and 'actual' data, certain adjustments can be made by the model to emulate 'mid-month' pumping program amendments. For example, when the actual inflow is much greater than the forecast inflow and pumping had commenced at the beginning of the time period, if the reservoir system reached capacity 'mid-month' the pumping is assumed to have stopped at the time the capacity of the system was reached.

In this mode, the operational decisions suggested by HOMA are based solely upon the information that would have been available to the operators at the time of the decision. The comparison therefore between the model and the real world operation of the system is valid.

- Planning - Perfect

The planning - perfect mode of HOMA is similar to the planning - forecast mode, however instead of using forecast data the model uses 'perfect' data, that is the 'actual' inflows and demands for the period being considered. In this way, HOMA is used to determine the best possible operating decisions for the system with the benefit of hindsight.

Although operators of the system would never be in this position, the results from model runs in this mode are none the less useful for the following reasons :

- They give a lower bound to the operating cost for the system.
- They give an indication of the potential benefits that can be made through improved forecasting techniques.
- They can be used to highlight certain optimum operating strategies.

Flow charts and further detailed description of the operation of HOMA in 'Planning - Forecast' and 'Planning - Perfect' modes have been presented previously by Crawley [48].

In the work presented within this thesis, HOMA has been used in 'Planning-Forecast' mode using the synthetic inflow, demand and pumping system failure records as input. The model has been used as a simulation tool for the system which incorporates an 'optimum' set of operating policies for the set of operating rules being examined.

4.3.1.1 Formulation of HOMA

An earlier version of HOMA was presented by Crawley [48] [50] as part of his Master of Engineering Science thesis. This model has been further enhanced with the inclusion of costs and capacities of water filtration plants and the facility for transfers between demand zones. This second feature is particularly important when considering reliability-cost tradeoffs for the metropolitan Adelaide headworks system. It enables the flexibility of transfers between demand zones to be included within the operation of the model. These transfers permit balancing of uneven storage distribution in the headworks system and in the case of the northern system, greater flexibility in the event of a component failure resulting in reduced capacity for one of the three pumping systems. The additional features that have been included within HOMA are outlined below.

The additional equations that have been used to include the additional model features are written in terms of general storages, as the formulation is applicable to any number of reservoirs for any multi-dimensional system.

The objective of HOMA is to determine sequences of optimal pumping, transfer and supply for the system, so as to minimise pumping, water filtration and demand zone transfer costs while satisfying a set of system operating rules.

The system operating rules consist of a series of minimum target storages for each reservoir for each of the time periods under consideration and an assumed set of forecast inflow and demand figures.

HOMA takes into account the non-linear nature of the pumping cost curve, the capacities of the pumping and pipeline network, the reservoir capacities, the transfer and spill capacities of each reservoir, the evaporation characteristics of each reservoir, the water filtration plant costs and capacities, and the demand zone transfer costs and capacities in determining the optimal operating policy.

Objective Function The full objective function used in HOMA is presented in Equation 4.1 and includes the water filtration plant and demand zone transfer costs undertaken as part of the research work outlined in this thesis.

Minimise :

$$
\begin{aligned}
Z \;=\; & \sum_{t=1}^{T}\sum_{n=1}^{N}(\alpha_t^n Q_t^n) \;+\; \sum_{t=1}^{T}\sum_{j=1}^{J}\sum_{c=1}^{C_j}(\beta_t^{jc} W_t^{jc}) \;+ \\
& \sum_{t=1}^{T}\sum_{n=1}^{N}(\gamma_t^o O_t^{on}) \;-\; \sum_{n=1}^{N}(\delta^n S_T^n) \;+ \\
& \sum_{t=1}^{T}\sum_{m=1}^{M}(\rho_t^m F_t^m) \;+\; \sum_{t=1}^{T}\sum_{x=1}^{Y}\sum_{y=1}^{Y}(\sigma^{xy} Z T_t^{xy}) \qquad (4.1)
\end{aligned}
$$

where,

c	=	Pump-cost curve slope change point
C_j	=	Total number of pump-cost curve coefficients for pump station (j)
F_t^m	=	Filtration plant (m) throughput during period (t)
j	=	Pump station number
J	=	Total number of pump stations
m	=	Filtration plant number
M	=	Total number of filtration plants
n, o	=	Reservoir number
N	=	Total number of reservoirs
O_t^{on}	=	Reservoir spill volume to reservoir (o) from reservoir (n) during period (t)
Q_t^n	=	Shortfall from target storage for reservoir (n) at the end of period (t)
S_T^n	=	Storage in reservoir (n) at the end of period (T)
t	=	Time period
T	=	Final time period

W_t^{jc} = Variables used to linearise pump-cost curve for pump station (j) during period (t)

x, y = Demand zone number

Y = Total number of demand zones

ZT_t^{xy} = Demand zone Transfer from demand zone (x) to demand zone (y) during period (t)

α_t^n = Shortfall penalty cost coefficient for reservoir (n) at the end of period (t)

β_t^{jc} = Pump-cost curve 'cost' coefficient for pump station (j) during period (t)

γ_t^o = Spill penalty coefficient for release to reservoir (o) during period (t)

δ^n = End of period benefit coefficient for for reservoir (n)

ρ_t^m = Filtration plant (m) 'variable cost' coefficient during period (t)

σ_t^{xy} = Demand zone transfer 'cost' coefficient associated with demand zone transfers from demand zone (x) to demand zone (y) during period (t)

The objective function seeks to minimise the sum of the costs for shortfalls below target storages $(\alpha_t^n Q_t^n)$, pumping electricity cost $(\beta_t^{jc} W_t^{jc})$, reservoir spillages costs $(\gamma_t^n O_t^{on})$, filtration plant variable costs $(\rho_t^m F_t^m)$, and demand zone transfer costs $(\sigma_t^{xy} ZT_t^{xy})$, while maximising the value of water in storage in the reservoirs at the end of the time period under consideration $(\delta^n S_T^n)$. Coefficients α_t^n, γ_t^n and δ^n are selected to reflect the relative importance of these various objectives.

Details of the objective function coefficients used within HOMA for the southern and northern systems are presented in Appendix B.

Constraint Equations The objective function given in Equation 4.1 is subject to a set of linear constraint equations. In addition to the constraint equations that have been described previously by Crawley [48] [50] and are presented in Appendix A, the following constraint equations have been added as part of the research work outlined in this thesis.

$$F_t^m \leq \chi_t^m \tag{4.2}$$

for $\mathrm{t} = 1, 2, \ldots, \mathrm{T}$; $\mathrm{m} = 1, 2, \ldots, \mathrm{M}$

where,

χ_t^m = Capacity of filtration plant (m) during time period (t)

$$ZT_t^{xy} \leq \varphi_t^{xy} \tag{4.3}$$

for $\mathrm{t} = 1, 2, \ldots, \mathrm{T}$; $\mathrm{x} = 1, 2, \ldots, \mathrm{X}$; $\mathrm{y} = 1, 2, \ldots, \mathrm{Y}$

where,

φ_t^{xy} = Capacity of the demand zone transfer from demand zone (x) to demand zone (y)

during time period (t)

$$D_t^m \;=\; F_t^m \;-\; ZT_t^{mx} \;-\; ZT_t^{my} \;+\; ZT_t^{xm} \;+\; ZT_t^{ym} \qquad (4.4)$$

for t = 1, 2, ..., T ; m = 1, 2, ..., M ; x = 1, 2, ..., X ; y = 1, 2, ..., Y

where,

D_t^m = Demand from Demand Zone (m) during period (t)

F_t^m = Filtration plant (m) throughput during period (t)

ZT_t^{mx} = Demand zone Transfer from demand zone (m) to demand zone (x) during period (t)

Equation 4.2 defines the water filtration plant capacity limitations for each water filtration plant.

Equation 4.3 defines the demand zone transfer capacity limitations between demand zones downstream of the water filtration plants.

Equation 4.4 defines continuity constraints for supply and transfer between demand zones. Apart from the pipeline on-line and lower north demand zones each demand zone (m) has an associated water filtration plant as highlighted in Figures 4.2 and 4.4.

Further details of the constraint equations used within HOMA for the southern and northern systems are presented in Appendix A. Details of the coefficients associated with the constraint equations used within HOMA for the southern and northern systems are presented in Appendix B.

4.3.1.2 Input Data to HOMA

The following sections describe the input data that is used in formulating the linear programming models within HOMA. This includes :

- Catchment Inflow Volumes
- System Demands
- Pump Cost Curves
- Water Filtration Plant Costs and Capacities
- Demand Zone Transfer Costs and Capacities
- Storage versus Evaporation Curves
- System Parameters

Catchment Inflow Volumes One objective of the work originally undertaken was to develop a model which can be used to assist in the operational decision-making process for the Adelaide headworks system.

The current procedure used to prepare the pumping program for the operation of the system involves the use of deterministic streamflow values obtained from statistical and hydrological analysis of the reservoir catchment streamflow records. Operating guidelines currently used by the EWS involve the use of forecast catchment inflows corresponding to an annual 70% probability of exceedance.

The forecast catchment inflows used by HOMA can be specified by the user and a range of inflow forecast sets have been considered in the examination of reliability-cost tradeoffs for the operation of the system.

Details of the catchment inflow exceedance values for both southern and northern systems are presented in Tables 4.3, 4.4, 4.5, 4.6, 4.7, 4.8, 4.9 and 4.10. The data contained in these tables has been obtained from analysis undertaken by the EWS of the historical records of reservoir inflows.

Month	90%	80%	70%	60%	50%	40%	30%	20%	10%
July	4.470	6.490	8.810	11.560	14.590	17.810	21.690	25.210	30.380
August	7.500	10.090	13.550	17.160	21.230	25.320	28.510	30.410	32.520
September	4.210	5.780	8.130	10.530	13.430	17.030	20.760	24.440	28.630
October	2.580	2.930	3.680	4.400	5.340	6.370	7.750	9.880	13.670
November	0.730	0.910	1.140	1.430	1.700	2.060	2.420	3.060	4.450
December	0.150	0.270	0.410	0.560	0.720	0.890	1.020	1.110	1.290
January	0.040	0.050	0.110	0.160	0.240	0.320	0.390	0.450	0.590
February	0.020	0.030	0.050	0.070	0.090	0.170	0.260	0.380	0.620
March	0.020	0.030	0.050	0.090	0.150	0.200	0.270	0.350	0.470
April	0.040	0.150	0.280	0.340	0.480	0.620	0.790	0.910	1.290
May	0.620	0.670	1.000	1.310	1.750	2.140	2.610	3.510	6.190
June	1.310	2.280	3.510	5.100	7.080	9.460	13.030	17.280	22.900
TOTAL	21.690	29.680	40.720	52.710	66.800	82.390	99.500	116.990	143.000

Table 4.3: Southern System - Mount Bold Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%	30%	20%	10%
July	0.810	1.400	1.950	2.500	3.000	3.550	4.400	6.100	10.100
August	0.860	1.450	2.200	3.200	4.100	5.000	6.000	7.000	8.500
September	0.520	0.750	1.000	1.300	1.700	2.300	3.100	4.400	7.050
October	0.320	0.500	0.620	0.750	0.880	1.100	1.400	1.900	3.150
November	0.200	0.260	0.320	0.390	0.450	1.530	0.630	0.770	1.020
December	0.100	0.170	0.220	0.260	0.300	1.340	0.390	0.460	0.550
January	0.080	0.140	0.190	0.230	0.250	0.280	0.300	0.330	0.390
February	0.080	0.130	0.160	0.190	0.220	0.240	0.260	0.290	0.360
March	0.070	0.120	0.170	0.210	0.250	0.280	0.320	0.370	0.450
April	0.150	0.220	0.270	0.320	0.360	0.400	0.460	0.540	0.710
May	0.280	0.370	0.420	0.480	0.530	0.630	0.800	1.100	2.050
June	0.340	0.540	0.800	1.100	1.450	1.900	2.500	3.400	6.300
TOTAL	3.810	6.050	8.320	10.930	13.490	16.550	20.560	26.660	40.630

Table 4.4: Southern System - Myponga Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	0.130	0.400	0.870	1.580	2.760	4.000
August	0.200	0.760	1.720	3.390	5.210	7.000
September	0.110	0.490	0.930	1.620	2.600	3.930
October	0.000	0.080	0.250	0.330	0.470	1.010
November	0.000	0.030	0.040	0.050	0.080	0.140
December	0.000	0.000	0.000	0.000	0.020	0.050
January	0.000	0.000	0.000	0.000	0.000	0.020
February	0.000	0.000	0.000	0.000	0.010	0.030
March	0.000	0.000	0.000	0.010	0.010	0.030
April	0.000	0.000	0.000	0.020	0.060	0.110
May	0.000	0.020	0.060	0.090	0.130	0.190
June	0.000	0.070	0.120	0.200	0.340	0.630
TOTAL	0.440	1.850	3.990	7.290	11.690	17.140

Table 4.5: Northern System - Warren Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	0.320	0.570	0.880	1.250	1.650	2.200
August	0.200	0.450	0.780	1.260	1.950	2.950
September	0.170	0.330	0.520	0.890	1.460	2.800
October	0.100	0.140	0.190	0.300	0.540	0.950
November	0.000	0.050	0.080	0.110	0.160	0.230
December	0.000	0.000	0.000	0.010	0.030	0.060
January	0.000	0.000	0.000	0.000	0.010	0.020
February	0.000	0.000	0.000	0.000	0.000	0.030
March	0.000	0.000	0.000	0.000	0.010	0.020
April	0.000	0.010	0.030	0.050	0.090	0.140
May	0.040	0.080	0.130	0.190	0.260	0.360
June	0.110	0.180	0.250	0.370	0.540	0.830
TOTAL	0.940	1.810	2.860	4.430	6.700	10.590

Table 4.6: Northern System - South Para Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	0.430	0.980	1.030	1.330	1.590	1.830
August	0.380	0.940	0.980	1.270	1.590	1.920
September	0.190	0.600	0.690	1.020	1.380	1.770
October	0.210	0.360	0.380	0.470	0.620	0.820
November	0.100	0.170	0.170	0.230	0.280	0.350
December	0.050	0.130	0.120	0.150	0.180	0.220
January	0.020	0.060	0.070	0.110	0.140	0.170
February	0.020	0.060	0.060	0.080	0.110	0.130
March	0.020	0.040	0.040	0.070	0.070	0.100
April	0.070	0.110	0.090	0.090	0.110	0.130
May	0.120	0.170	0.140	0.160	0.170	0.180
June	0.170	0.280	0.230	0.270	0.320	0.390
TOTAL	1.780	3.900	4.000	5.250	6.560	8.010

Table 4.7: Northern System - Little Para Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	1.241	1.940	2.818	3.874	5.135	6.521
August	1.927	3.419	4.917	6.712	8.455	10.421
September	1.432	2.475	3.432	4.607	5.973	7.735
October	0.799	1.340	1.795	2.198	2.396	2.541
November	0.350	0.515	0.667	0.772	0.851	0.970
December	0.119	0.198	0.277	0.343	0.396	0.462
January	0.000	0.073	0.132	0.172	0.205	0.238
February	0.000	0.033	0.066	0.086	0.119	0.132
March	0.000	0.006	0.026	0.059	0.092	0.132
April	0.026	0.092	0.178	0.231	0.271	0.310
May	0.185	0.290	0.409	0.508	0.601	0.772
June	0.521	0.772	1.056	1.426	1.907	2.765
TOTAL	6.600	11.153	15.773	20.988	26.401	32.999

Table 4.8: Northern System - Millbrook Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	0.301	0.470	0.683	0.939	1.245	1.581
August	0.467	0.829	1.192	1.627	2.050	2.526
September	0.347	0.600	0.832	1.117	1.448	1.875
October	0.194	0.325	0.435	0.533	0.581	0.616
November	0.085	0.125	0.162	0.187	0.206	0.235
December	0.029	0.048	0.067	0.083	0.096	0.112
January	0.000	0.018	0.032	0.042	0.050	0.058
February	0.000	0.008	0.016	0.021	0.029	0.032
March	0.000	0.002	0.006	0.014	0.022	0.032
April	0.006	0.022	0.043	0.056	0.066	0.075
May	0.045	0.071	0.099	0.123	0.146	0.187
June	0.126	0.187	0.256	0.346	0.462	0.670
TOTAL	1.600	2.705	3.823	5.088	6.401	7.999

Table 4.9: Northern System - Kangaroo Creek Inflow Exceedance Volumes (GL)

Month	90%	80%	70%	60%	50%	40%
July	0.338	0.530	0.769	1.057	1.400	1.778
August	0.526	0.932	1.341	1.831	2.306	2.842
September	0.391	0.675	0.936	1.256	1.629	2.110
October	0.218	0.365	0.490	0.599	0.653	0.693
November	0.095	0.140	0.182	0.211	0.232	0.265
December	0.032	0.054	0.067	0.094	0.108	0.126
January	0.000	0.019	0.036	0.047	0.056	0.065
February	0.000	0.009	0.018	0.023	0.032	0.036
March	0.000	0.002	0.007	0.016	0.025	0.036
April	0.007	0.026	0.049	0.063	0.074	0.085
May	0.050	0.079	0.112	0.139	0.164	0.211
June	0.142	0.211	0.288	0.389	0.520	0.754
TOTAL	1.799	3.042	4.304	5.725	7.199	9.001

Table 4.10: Northern System - Hope Valley Inflow Exceedance Volumes (GL)

System Demands The forecast system demands used in HOMA are deterministic in nature and can be specified by the user. Operating guidelines currently used by the EWS involve the use of forecast demands corresponding to the average demand for the last five years. Using these forecast demands together with a set of forecast inflow volumes, a set of monthly pumping and transfer decisions are made for the operation of the system.

Flexibility within the system downstream of the water filtration plants allows these demands to be met from several sources constrained only by the demand zone transfer capacities.

Details of the current five year average demand values used in the HOMA runs in this thesis are presented in Tables 4.11 and 4.12.

Month	Happy Valley Demand Zone (SDHV) (GL)	Myponga Demand Zone (SDMY) (GL)	Encounter Bay Demand (SDEB) (GL)	Pipeline Online Demand (PMBOO) (GL)	Southern System Total Demand (GL)
July	3.610	0.230	0.100	0.390	4.330
August	3.670	0.230	0.100	0.450	4.450
September	4.180	0.270	0.100	0.500	5.050
October	5.760	0.370	0.200	0.600	6.930
November	8.070	0.520	0.300	0.780	9.670
December	9.060	0.580	0.400	0.900	10.940
January	10.780	0.690	0.500	1.010	12.980
February	9.400	0.600	0.400	0.940	11.340
March	9.310	0.590	0.300	0.890	11.090
April	6.240	0.400	0.200	0.580	7.420
May	4.460	0.280	0.100	0.410	5.250
June	3.510	0.220	0.100	0.360	4.190
TOTAL	78.050	4.980	2.800	7.810	93.640

Table 4.11: Southern System - Forecast Demand Volumes

Month	Lower North Demand (NDWR) (GL)	Barossa Demand Zone (NDBR) (GL)	Little Para Demand Zone (NDLP) (GL)	Anstey Hill Demand Zone (NDAH) (GL)	Hope Valley Demand Zone (NDHV) (GL)	Mannum-Adelaide Online Demand (PMAO) (GL)
July	0.355	1.290	0.090	1.710	1.780	0.120
August	0.335	1.300	0.000	1.610	1.770	0.100
September	0.295	1.420	0.000	1.730	2.070	0.100
October	0.435	1.480	0.640	2.190	2.700	0.110
November	0.895	1.480	1.540	3.230	3.630	0.190
December	1.125	1.740	1.850	3.620	3.970	0.210
January	1.495	2.150	2.410	4.530	4.420	0.300
February	1.265	1.880	1.900	3.900	3.930	0.270
March	1.135	1.850	1.940	3.960	4.020	0.260
April	0.875	1.230	1.290	2.760	2.910	0.160
May	0.485	1.720	0.270	1.770	2.070	0.100
June	0.385	1.120	0.000	1.710	1.740	0.080
TOTAL	9.080	18.660	11.930	32.720	35.010	2.000

Table 4.12: Northern System - Forecast Demand Volumes

Pump Cost Curves Pumping from the River Murray to the metropolitan Adelaide headworks system can be carried out through three major pumping/pipeline systems together with the Millbrook pump station. Each of these systems consist of a number of pumps that can be run individually or in parallel.

The pumps are electrically powered and the EWS is charged for the power consumed by the Electricity Trust of South Australia (ETSA). ETSA offers a special seasonal electricity tariff to the EWS for pumping on the major pipeline systems and at Millbrook pump station. This tariff schedule comprises off-peak and on-peak periods for the winter, spring/autumn, and summer seasons.

The combination of the number of pumps and arrangement of the electricity tariff periods, results in a non-linear pump-cost curve. All combinations of pumps and electricity tariff periods are considered, from no pumps running to all pumps running continuously. A lower bound to the series of points plotted is taken and the curve converted into a piece-wise linear function. These simplifications enable convex pumping-cost curves to be included in the linear programming formulation. Details of the application of piece-wise linearisation to the Adelaide headworks system pump cost curves has been previously described by Crawley [48] [50].

The effect of component failures on the operation of the pumping systems have been included through modification of the pumping cost curves. These modifications are described in detail later in this thesis.

The pump-cost curves used in this study for the three major pumping systems and the Millbrook pump station, when fully operational, are presented in Appendix B.

Water Filtration Plant Costs and Capacities Integral to the operation of the Adelaide water supply headworks system are six water filtration plants. Costs associated with the operation of these water filtration plants can be considered in terms of fixed and variable costs.

Fixed costs include labour, materials, plant and vehicle hire, administration, training and safety expenses which are minimally affected by the quantity of water treated. Variable costs include electricity and chemicals which are directly related to the quantity of water treated through the plant. In the HOMA model only the variable costs have been included.

The extension of HOMA to include water filtration plant costs and capacities has been undertaken as part of the work detailed in this thesis.

The water filtration plant costs and capacities used in this study for the six metropolitan Adelaide water filtration plants are presented in Appendix B.

Demand Zone Transfer Costs and Capacities Adelaide's consumers receive water from a reticulation system comprising 120 tanks, 48 pumping stations and a complex network of over 8000 km of water mains varying in size from 2100 mm to 75 mm in diameter. The headworks system is connected to the reticulation system by a series of trunk mains. These trunk mains deliver water to supply tanks located throughout the reticulation system. From these tanks, distribution mains are used to supply consumers. Hydraulically operated valves regulate the flow into these tanks from the trunk mains. The majority of the tanks are gravity fed, however in certain locations, distribution pump stations are utilised.

As previously highlighted in Figure 4.6, water can be supplied to certain areas from more than one water filtration plant. This flexibility has been included in the optimisation model for the operation of the headworks system. When seeking optimal operation of the system, variations in water filtration plant costs can be exploited to minimise the overall cost of the operation of the system. When examining reliability-cost tradeoffs for the operation of the system, this flexibility enhances the reliability for a given set of operating rules. The extension of HOMA to include demand zone transfer capacities and costs has been undertaken as part of the work detailed in this thesis.

Details of the assumed demand zone transfer costs and capacities adopted in this study are presented in Appendix B.

Storage versus Evaporation Curves For each reservoir, evaporation losses can be plotted against storage volume for each time period and a linear regression applied to these plotted points.

Details of the inclusion of the evaporation loss curves in HOMA have been previously presented by Crawley [48] [50].

The evaporation loss curves for the ten reservoirs in the Adelaide headworks system adopted in this study are presented in Appendix B.

System Parameters The remaining constraint equations used in the linear program formulations for the southern and northern systems in HOMA have previously been described by Crawley [48] [50]. These constraint equations include :

- Constraints on the Mannum-Adelaide pipeline
- Constraints on the Swan Reach-Stockwell pipeline
- Constraints on certain reservoir intakes

Details of these constraints have been presented earlier in this chapter.

4.3.2 Summary

In this section, a description of the Headworks Optimisation Model - Adelaide (HOMA) is detailed. HOMA is currently used in 'operational' mode by the pumping engineer within the EWS to assist in the planning and operation of the headworks system.

Additional features of HOMA have been developed within the work detailed in this thesis. These additional features are outlined and include cost and capacities of water filtration plants, and the costs and capacities of transfers between demand zones downstream of the water filtration plants.

HOMA, described in this section, has been used as a simulation/optimisation tool within the research work presented in this thesis, in the consideration of the reliability-cost tradeoffs for the Adelaide headworks system.

4.4 Synthetic Inflow Data Generation

In the operation of an urban water supply reservoir system, decisions must be made regarding the reservoir transfer and release volumes, and the imposition of restrictions on the system. These decisions are based on the current levels of storage in the reservoirs in the system and an assessment of the likely future inflows and demands into and out of the system.

In order to examine risk and reliability aspects of a water supply headworks system it is necessary to consider the operation of the system over a sufficiently long period of time to obtain statistically valid results. Historical records for the system are of limited length and in order to assess the long-term performance of the system it is helpful to generate synthetic data that can be used as input to a system optimisation model. In this section, a synthetic multisite streamflow data generation model is described that has been developed for the Adelaide water supply headworks system. Data generated using this model has been used to test the long-term performance of the Adelaide headworks system under a range of operating rules.

4.4.1 Background to the Multisite Streamflow Data Generation Model

Work undertaken within the Department of Civil and Environmental Engineering at the University of Adelaide, has examined the use of multisite and single site time series models for the short term forecasting of streamflows. The work included the development of a first-order multisite multiperiod autoregressive data generation model of a similar type to the model developed by Young and Pisano [329].

The model developed by Baker and Dandy [9] [10] [64] is given by Equation 4.5.

$$[Z_t] = [A_t][Z_{t-1}] + [B_t][\epsilon_t] \tag{4.5}$$

where,

$[Z_t]$ = a (n x 1) vector of transformed, standardised monthly flow or rainfall data at n sites

$[A_t]$ = a (n x n) matrix of coefficients

$[B_t]$ = a (n x n) matrix of coefficients

$[A_t]$ and $[B_t]$ may be determined using Equations 4.6 and 4.7.

$$[A_t] = [M_{1,t}][M_{0,t-1}]^{-1} \tag{4.6}$$

$$[B_t][B_t]^T = [M_{0,t}] - [M_{1,t}][M_{0,t-1}]^{-1}[M_{1,t}]^T \tag{4.7}$$

where,

ϵ_t = a (n x 1) vector of $N(0,1)$ random variables

$[M_{0,t}]$ = a (n x n) matrix of lag zero cross-correlation coefficients between sites for month (t)

$[M_{1,t}]$ = a (n x n) matrix of lag one cross-correlation coefficients between sites for month (t)

The generation equation given in Equation 4.5 uses standardised, transformed values. The generated values must be 'shaped' to resemble the historical form by backtransformation. A three parameter log transformation has been adopted in the model developed by Baker and Dandy for generation of synthetic inflow data for the Adelaide system. The three parameter log transformation equations are given in Equations 4.8 and 4.9.

$$y_t = ln(x_t - \tau_t) \tag{4.8}$$

$$z_t = \frac{y_t - \mu_t}{\sigma_t} \tag{4.9}$$

where,

x_t = flow or rainfall value for month (t)

y_t = transformed flow or rainfall value for month (t)

z_t = transformed and standardised flow or rainfall value for month (t)

τ_t = a monthly shift parameter for each series, chosen so as to give a zero skewness in y_t

μ_t = mean of y_t

σ_t = standard deviation of y_t

This data generation model has been described in detail by Baker and Dandy [9] [10] [64].

As the size of a model of this form increases, there is a tendency toward mathematically instability in estimating the coefficients contained in matrices $[A_t]$ and $[B_t]$. For this reason, a six station model was adopted for application to the Adelaide Hills catchments using five streamflow sites and one rainfall station. Using this model, serial flow correlations at individual sites and spatial correlations between sites have been utilised in the generation of synthetic streamflow and rainfall data.

The location of these stations is shown in Figure 4.18 and details of the streamflow and rainfall sites record periods and data set lengths are given in Table 4.13.

Streamflow Gauging Stations			
Station Location	Station Identifier	Record Period	Record Length (Years)
Myponga	G.S. 502 501	1934-1984	51
Onkaparinga (at Clarendon Weir)	G.S. 503 500	1898-1984	87
Gorge Weir	G.S. 504 501	1884-1983	100
Gumeracha Weir	G.S. 504 500	1918-1983	66
South Para (at Barossa Weir)	G.S. 505 501	1939-1980	42
Rainfall Gauging Station			
Millbrook Reservoir	R.F. 023 731	1914-1988	75

Table 4.13: Gauging Station Record Periods and Lengths

The streamflow values contained in these records, although referred to as natural inflows, are in reality reconstructed. These figures are based on gauged values at the respective sites and water balance equations composed of variables that seek to remove the human induced effects on the system. The water balance equations for some of the sites have up to twelve variables and include influences such as reservoir evaporation, changes in storage and volumes

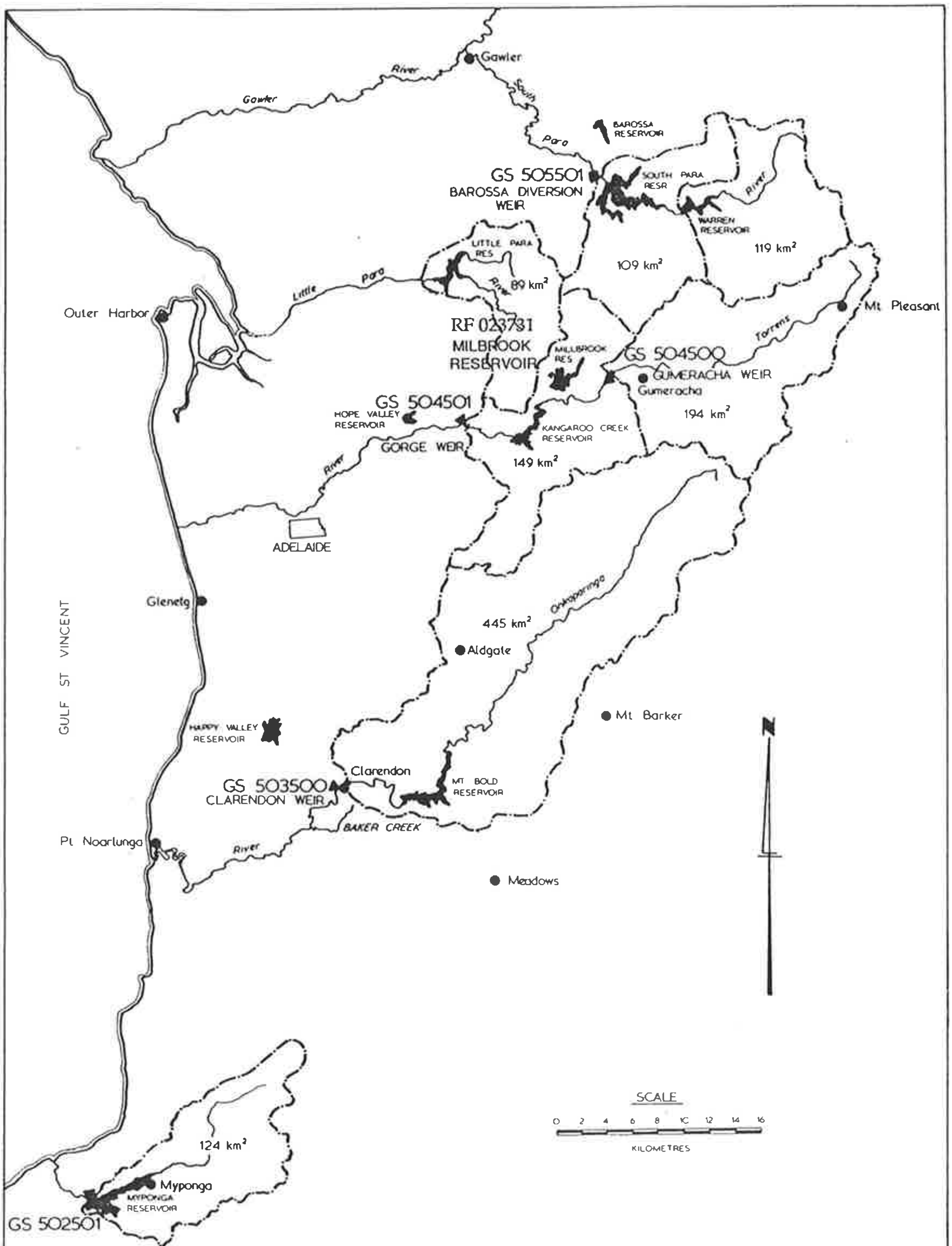


Figure 4.18: Streamflow and Rainfall Gauging Station Locations

pumped from the River Murray.

As the individual components contain measurement and other errors, the flow data also contains errors which influence the results to an unknown extent. When changes in storage levels are used to determine reservoir inflows, these water balance equations will involve the difference between terms of similar magnitude. In such cases the errors are magnified as a percentage of the final estimated value. The effect of the water balance equations is clear for some data, such as the South Para data set, where negative inflows occur with regularity. Baker [10] noted that almost all of the streamflow data sets contained some missing data, but for any particular month, less than five percent of the values were missing. These missing values were replaced by mean values for the particular month determined from the remaining values. Baker concluded that the general quality of the streamflow data was quite varied with the Myponga data set being least affected by errors, Onkaparinga, Gorge Weir and Gumeracha Weir records being of only acceptable quality while the South Para record was of poor quality.

Baker considered that the rainfall record considered was of high quality and produced consistently good results.

The model developed by Baker and Dandy can be used to generate monthly synthetic streamflow data for these five sites and involves a total of 1080 parameters, comprising mean, variance and a 'shifting parameter' for each of the sites for each month together with the monthly coefficients of the $[A]$ and $[B]$ matrices. Although a rigorous examination of the effect of the uncertainty of each of these parameters would be ideal, this is considered practically not possible. As an alternative, a sensitivity analysis is proposed for those parameters identified as having the largest impact on the reliability-cost tradeoffs for the system. Details of this parameter sensitivity analysis is presented in Section 5.2.3 of this thesis.

4.4.2 Inflow Data Generation

Using the multisite model, synthetic inflow data for the southern and northern subsystems of the Adelaide Headworks system were generated. The length of generated data was 10,000 years. Only a portion of this data has been used in this research when examining the performance of operating policies for the system.

More detailed summary statistics for the historical and generated data are presented in Appendix E.

4.4.2.1 Southern System Gauging Stations

Inflow data has been generated at two inflow gauging stations for the southern system. These stations are Myponga and Onkaparinga (at Clarendon Weir) and are located as shown on Figure 4.18.

Myponga Gauging Station The historical and generated inflow statistics for the Myponga gauging station are shown in Table 4.14.

Onkaparinga (at Clarendon Weir) Gauging Station The historical and generated inflow statistics for the Onkaparinga gauging station at Clarendon Weir are shown in Table 4.15.

Month	Mean		Median	
	Historical (ML)	Generated (ML)	Historical (ML)	Generated (ML)
January	300.6	295.1	264.0	231.7
February	304.9	285.3	232.0	208.3
March	255.2	256.2	261.5	237.1
April	404.7	405.1	369.0	360.8
May	997.8	1029.1	625.0	655.7
June	2594.1	2795.7	1262.0	1546.3
July	4391.4	4542.7	3168.5	3283.2
August	4480.7	4554.3	4124.5	3808.8
September	3054.5	3182.4	2010.0	1978.5
October	1478.6	1532.2	898.0	1053.0
November	580.1	587.0	541.0	500.2
December	320.3	321.1	297.0	320.1
	Standard Deviation		Coefficient of Skewness	
	Historical (ML)	Generated (ML)	Historical	Generated
January	331.3	245.8	4.168	1.897
February	506.2	263.8	5.532	2.453
March	144.5	146.4	0.441	0.716
April	243.0	231.0	1.805	1.220
May	1045.0	1163.9	2.271	3.272
June	2803.1	3914.2	1.887	3.361
July	3685.0	4441.8	1.196	2.547
August	3213.1	3445.8	0.969	1.466
September	3082.3	3775.7	1.827	3.109
October	1395.7	1586.1	1.795	2.725
November	352.8	364.7	1.351	1.575
December	165.0	163.7	-0.051	-0.032

Table 4.14: Myponga Inflow Historical and Generated Statistics

Month	Mean		Median	
	Historical (ML)	Generated (ML)	Historical (ML)	Generated (ML)
January	366.7	374.2	310.5	327.7
February	514.8	491.1	271.5	236.8
March	269.8	267.8	231.5	253.4
April	1079.9	971.6	524.0	515.1
May	3543.5	3740.0	1830.0	1811.8
June	8664.8	9309.7	3257.5	3871.6
July	15857.7	16367.0	13393.0	11409.5
August	18079.3	18695.4	15874.5	13980.7
September	11996.8	12554.4	7964.5	8693.9
October	7605.7	8329.6	4601.0	4590.7
November	2794.9	2835.8	1366.0	1817.8
December	826.9	827.4	708.0	721.5
	Standard Deviation		Coefficient of Skewness	
	Historical (ML)	Generated (ML)	Historical	Generated
January	663.9	639.9	1.368	0.433
February	1132.7	849.4	3.500	2.388
March	295.3	299.2	0.205	0.222
April	2595.2	1459.7	5.473	3.463
May	5464.3	6331.5	2.968	3.952
June	12919.2	17169.2	2.557	3.951
July	14638.7	16917.9	1.652	2.512
August	14387.5	16705.1	1.148	2.020
September	10252.7	13011.8	1.066	2.663
October	7979.4	11908.9	1.524	3.661
November	3020.9	3332.7	1.850	2.910
December	704.9	691.9	1.019	0.904

Table 4.15: Onkaparinga Inflow Historical and Generated Statistics

4.4.2.2 Northern System Gauging Stations

Inflow data has been generated at three inflow gauging stations and one rainfall gauging station for the northern system. These stations are South Para, Gorge Weir, Gumeracha Weir and Millbrook Reservoir rainfall station and are located as shown on Figure 4.18.

South Para Gauging Station The historical and generated inflow statistics for the South Para gauging station are shown in Table 4.16.

Month	Mean Historical (ML)	Mean Generated (ML)	Median Historical (ML)	Median Generated (ML)
January	67.7	69.1	2.5	24.2
February	118.6	120.4	10.5	51.1
March	40.8	43.0	23.5	42.8
April	216.8	220.1	159.5	158.1
May	1110.4	1220.2	365.5	417.0
June	3548.7	4380.9	678.0	955.8
July	6024.0	6589.1	4096.5	3406.8
August	7937.6	8837.6	4987.5	4757.7
September	5633.9	6268.2	3476.5	3359.8
October	2912.2	3556.9	1621.5	1292.0
November	716.2	717.3	258.0	362.9
December	142.9	144.1	70.0	91.8
	Standard Deviation Historical (ML)	Standard Deviation Generated (ML)	Coefficient of Skewness Historical	Coefficient of Skewness Generated
January	239.4	220.5	1.946	1.246
February	288.6	284.2	1.538	1.693
March	158.3	158.3	0.014	-0.015
April	271.3	270.3	1.563	1.622
May	2012.6	2672.1	3.307	4.331
June	5841.4	11814.0	1.745	4.821
July	7130.0	10219.5	1.893	3.471
August	8233.8	12961.2	1.137	3.382
September	5876.8	9241.6	1.386	3.363
October	3905.0	7707.5	2.086	4.758
November	1136.6	1104.4	2.761	2.847
December	238.9	238.0	1.339	1.386

Table 4.16: South Para Inflow Historical and Generated Statistics

Gorge Weir Gauging Station The historical and generated inflow statistics for the Gorge Weir gauging station are shown in Table 4.17.

Month	Mean		Median	
	Historical (ML)	Generated (ML)	Historical (ML)	Generated (ML)
January	314.5	319.7	258.0	248.4
February	255.7	256.1	166.0	120.0
March	75.6	675.2	111.0	87.3
April	496.4	507.0	345.5	361.4
May	1864.4	1910.8	986.0	1106.3
June	5150.9	6352.9	1542.5	1827.8
July	9643.6	10161.8	5237.5	6000.2
August	11716.9	12465.7	8332.5	7976.7
September	8290.7	8820.3	4752.5	5620.4
October	4933.3	5298.0	3095.0	2951.7
November	1782.2	1813.0	1068.0	1272.1
December	689.0	694.2	512.0	559.6
	Standard Deviation		Coefficient of Skewness	
	Historical (ML)	Generated (ML)	Historical	Generated
January	471.4	461.3	1.188	0.970
February	720.0	642.1	2.869	1.391
March	358.3	3269.8	-0.226	6.392
April	727.0	628.2	3.208	1.411
May	2571.6	2588.9	2.656	3.200
June	7726.1	16031.2	2.107	4.672
July	10541.5	13503.6	1.668	3.096
August	10664.8	14564.2	0.827	2.703
September	7672.6	10542.3	0.866	2.819
October	5822.7	7393.7	1.991	3.386
November	1833.0	1919.6	1.898	2.425
December	708.5	670.2	2.047	1.239

Table 4.17: Gorge Weir Inflow Historical and Generated Statistics

Gumeracha Weir Gauging Station The historical and generated inflow statistics for the Gumeracha Weir gauging station are shown in Table 4.18.

Month	Mean Historical (ML)	Mean Generated (ML)	Median Historical (ML)	Median Generated (ML)
January	121.1	120.9	64.5	95.4
February	148.2	137.7	46.0	69.8
March	96.4	96.0	53.0	81.7
April	142.9	143.7	68.5	78.7
May	652.3	842.0	207.0	228.5
June	2390.1	3241.3	571.0	732.5
July	4460.5	4963.7	1769.0	2494.3
August	6086.7	6599.3	4680.5	3915.4
September	4000.7	4417.2	2259.0	2441.1
October	2220.9	2510.2	1139.5	1141.9
November	624.7	639.0	345.0	403.4
December	224.5	222.9	143.5	136.4
	Standard Deviation Historical (ML)	Standard Deviation Generated (ML)	Coefficient of Skewness Historical	Coefficient of Skewness Generated
January	179.5	150.3	2.951	1.094
February	345.2	241.1	3.762	2.202
March	115.6	111.5	1.224	0.750
April	203.7	207.7	2.505	3.299
May	1168.7	2479.5	3.552	5.024
June	3832.8	9745.6	1.922	5.090
July	4787.9	7882.0	1.118	3.677
August	5964.4	8502.7	1.177	3.036
September	3914.6	6282.4	0.900	3.352
October	2769.1	4300.8	1.987	3.956
November	713.0	773.2	2.143	2.849
December	356.8	287.1	3.824	2.451

Table 4.18: Gumeracha Weir Inflow Historical and Generated Statistics

Millbrook Reservoir Rainfall Gauging Station The historical and generated rainfall statistics for the Millbrook Reservoir rainfall gauging station are shown in Table 4.19.

Month	Mean Historical (mm)	Mean Generated (mm)	Median Historical (mm)	Median Generated (mm)
January	28.19	29.46	16.35	19.06
February	33.49	35.31	20.95	19.71
March	26.69	29.33	13.05	15.01
April	70.22	71.45	68.05	59.21
May	109.86	111.82	93.40	95.79
June	97.43	97.44	93.85	85.02
July	127.08	128.19	128.70	123.21
August	111.15	112.64	102.70	106.07
September	94.07	95.36	79.90	83.99
October	81.86	82.45	71.80	71.25
November	51.72	51.41	45.50	46.52
December	36.26	36.11	33.35	31.88
	Standard Deviation Historical (mm)	Standard Deviation Generated (mm)	Coefficient of Skewness Historical	Coefficient of Skewness Generated
January	26.75	35.07	1.256	2.885
February	36.94	48.63	1.710	3.028
March	29.05	45.72	1.553	3.435
April	51.73	55.50	0.886	1.435
May	66.96	73.53	0.604	1.511
June	59.11	62.81	0.775	1.274
July	51.92	53.82	0.123	0.534
August	49.90	51.32	0.504	0.708
September	52.65	56.13	0.653	1.323
October	52.31	55.30	0.764	1.360
November	29.99	30.29	0.778	1.001
December	23.17	23.69	0.678	1.076

Table 4.19: Millbrook Reservoir Rainfall Historical and Generated Statistics

Over the last 20 years, the metropolitan Adelaide water supply system has changed to accommodate the changing demands for water supply. These changes have included both temporal and quantitative changes. The purpose of the work presented in this thesis is to examine the current reliability of the system rather than the historical reliability or future reliability of the system. The tools developed however, will be useful in examining the future reliability of the system, given certain assumptions concerning future trends in system demands or even changes to the reservoir inflows due to say climatic changes.

4.4.3 Flow Frequency Analysis of Generated and Historical Inflow Data

It is important that the data generated using the synthetic inflow generation model reflects the same statistical properties as the historical data set on which it is based. It must be noted that the generated data attempts to match the historical data on a monthly basis. Of importance to the reliability assessment of the Adelaide water supply headworks system is not only the accuracy of monthly inflow volumes, but also the low flow events having durations of six months, one, two, five and ten years, with the critical periods being of two, five and ten years duration.

A range of methods have been proposed by McMahon and Mein [203] for the determination of flow duration curves from a streamflow record. In this subsection, one method suggested by McMahon and Mein [203] to determine the flow duration curves for both the generated and historical data sets will be described. For durations greater than or equal to one year, a different method has been used than for durations less than one year.

4.4.3.1 Analysis for Flow Durations less than one year

For low flow events of duration less than one year, the following method has been used to determine the low flow frequency curves for both the historical data and the synthetically generated data.

For each of the n month periods under consideration, the minimum consecutive n month flow for each water year in the record is determined. The water year commences at the beginning of July and ends at the end of June in the following year. These are ranked with the lowest flow being ranked 1. Plotting positions are assigned to each flow value in terms of recurrence interval using Equation 4.10.

$$p = \frac{M - c}{N - 2c + 1} \tag{4.10}$$

where,

p = Sample Probability in any year of an n-month flow being equal to or less than the recorded value

M = Rank of the recorded n-consecutive monthly sequence

c = Distribution type constant

N = Number of years in the record

The constant c varies according to the theoretical distribution type assumed. For the normal distribution $c = \frac{3}{8}$. If the applicable distribution is unknown then Cunnane [55] proposed the best value to use is 0.4.

4.4.3.2 Analysis for Flow Durations greater than or equal to one year

For low flow events approaching, or in excess of 12 months duration, the following method proposed by Hudson and Roberts [151] has been used. Streamflows of n consecutive months are selected such that overlap does not occur. This is achieved by searching the record for the lowest n-month sequence; this is given rank 1. The remaining portions of the record are searched for the next lowest sequence of n-consecutive months; this is given rank 2. And so on until there is no longer an unbroken n-sequence left. The series formed is termed the non-overlapping or independent series.

Once the flows have been ranked, sample recurrence intervals are attached to each flow value using Equation 4.11.

$$T = \frac{N - 2c + 1}{M - c} \tag{4.11}$$

With respect to the duration of the event under consideration (n months in duration), the average recurrence interval of an n-month event (T_n) of rank M is obtained using Equation 4.12.

$$T_n = \frac{(\frac{12N}{n}) - 2c + 1}{M - c} \tag{4.12}$$

where,

$\frac{12N}{n}$ = The maximum number of independent events in the sequence.

The recurrence interval T_n has units of n months and is converted to years using Equation 4.13

$$T = \frac{n}{12} T_n \tag{4.13}$$

4.4.3.3 Comparison of Historical and Generated Cumulative Flow Frequency Curves

The methodology described in the previous sections has been applied to the synthetically generated and historical data sets for the following streamflow sites : Myponga, Onkaparinga, South Para, and Gorge Weir. Using the historical record set together with the synthetically generated inflow record of 1000 years in length, cumulative flow frequency curves for periods ranging from 2 months to 10 years have been determined. The results from this analysis are shown graphically in Figures 4.19, 4.20, 4.21, 4.22, and 4.23.

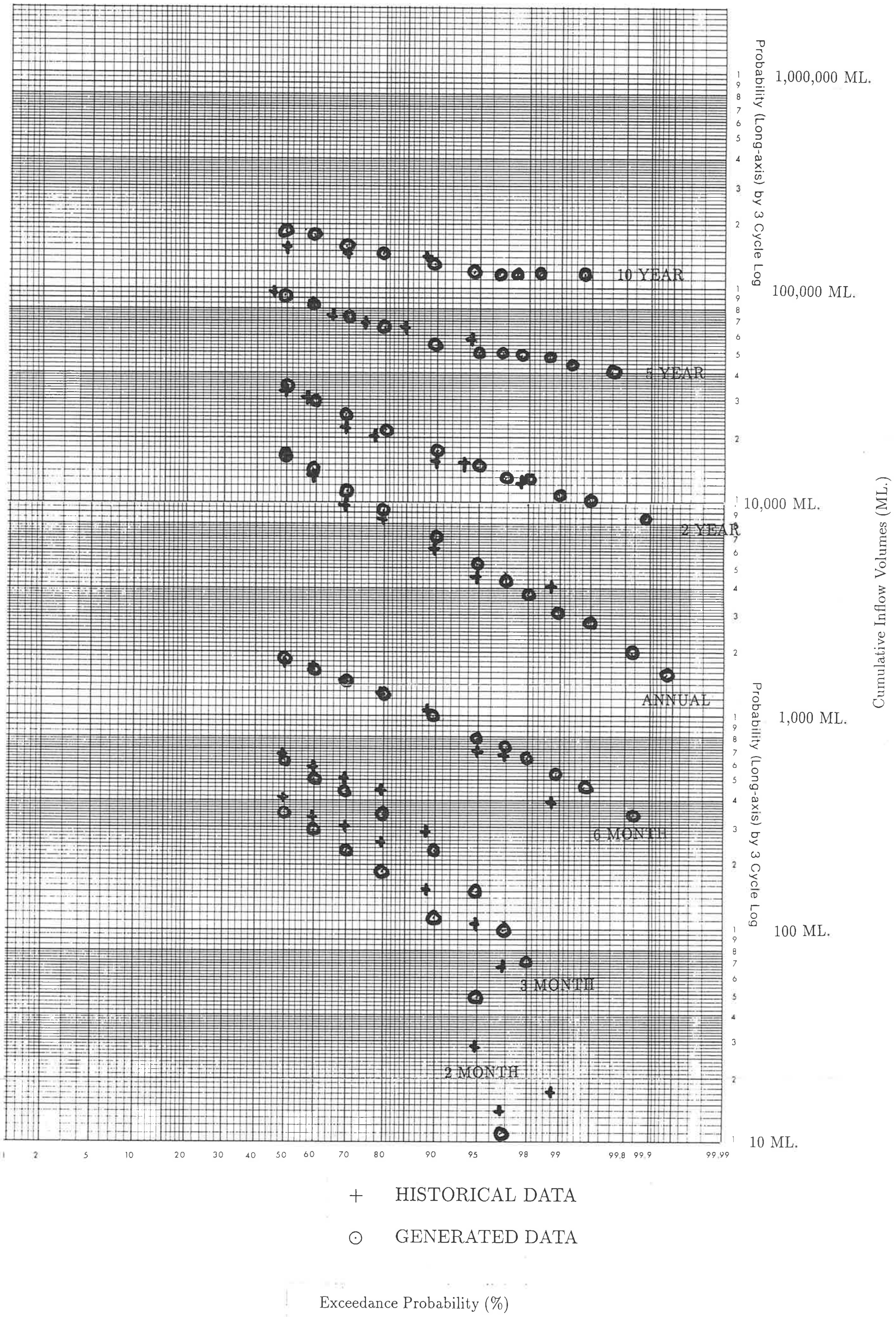


Figure 4.19: Myponga Cumulative Flow Frequency Curves

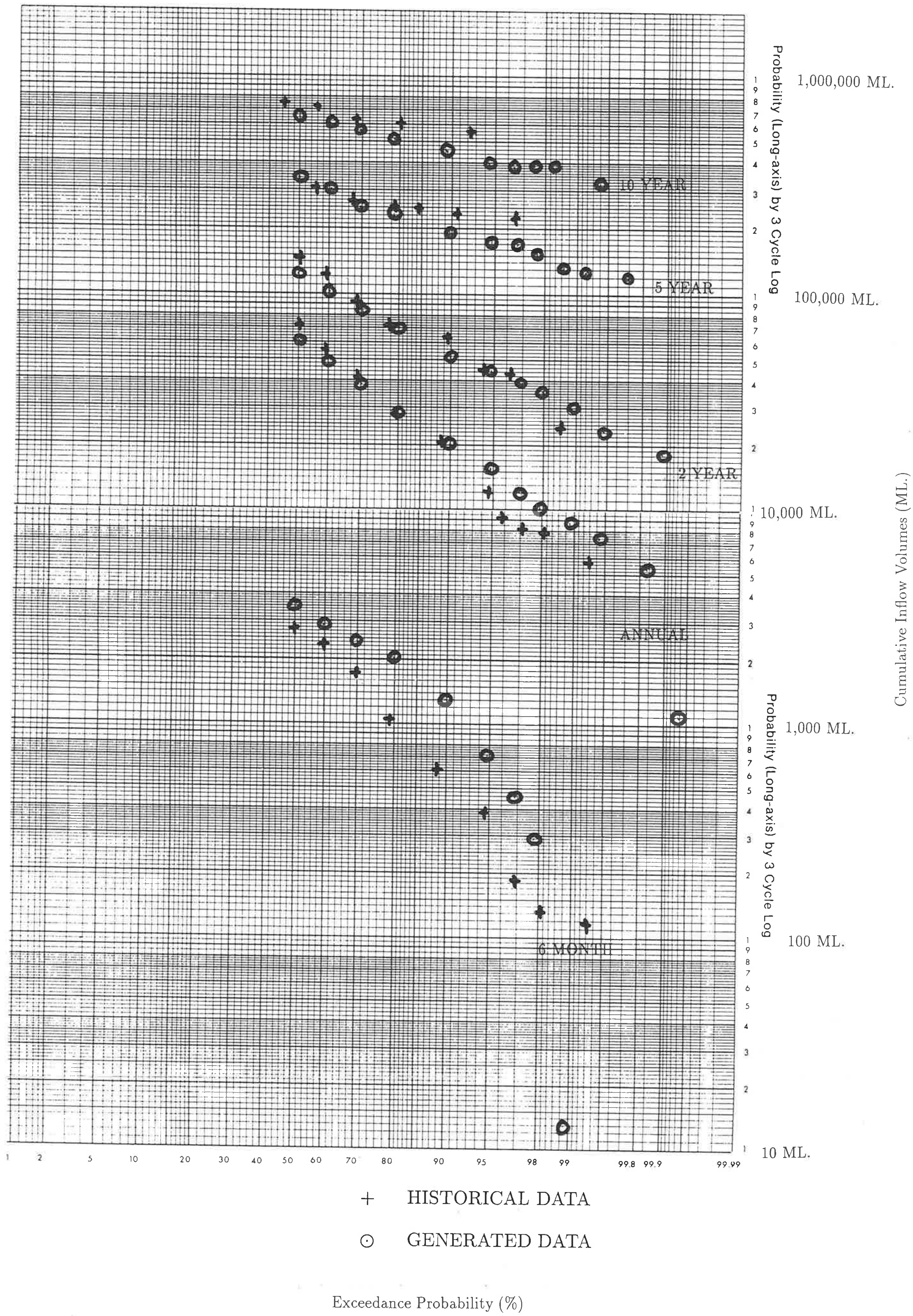
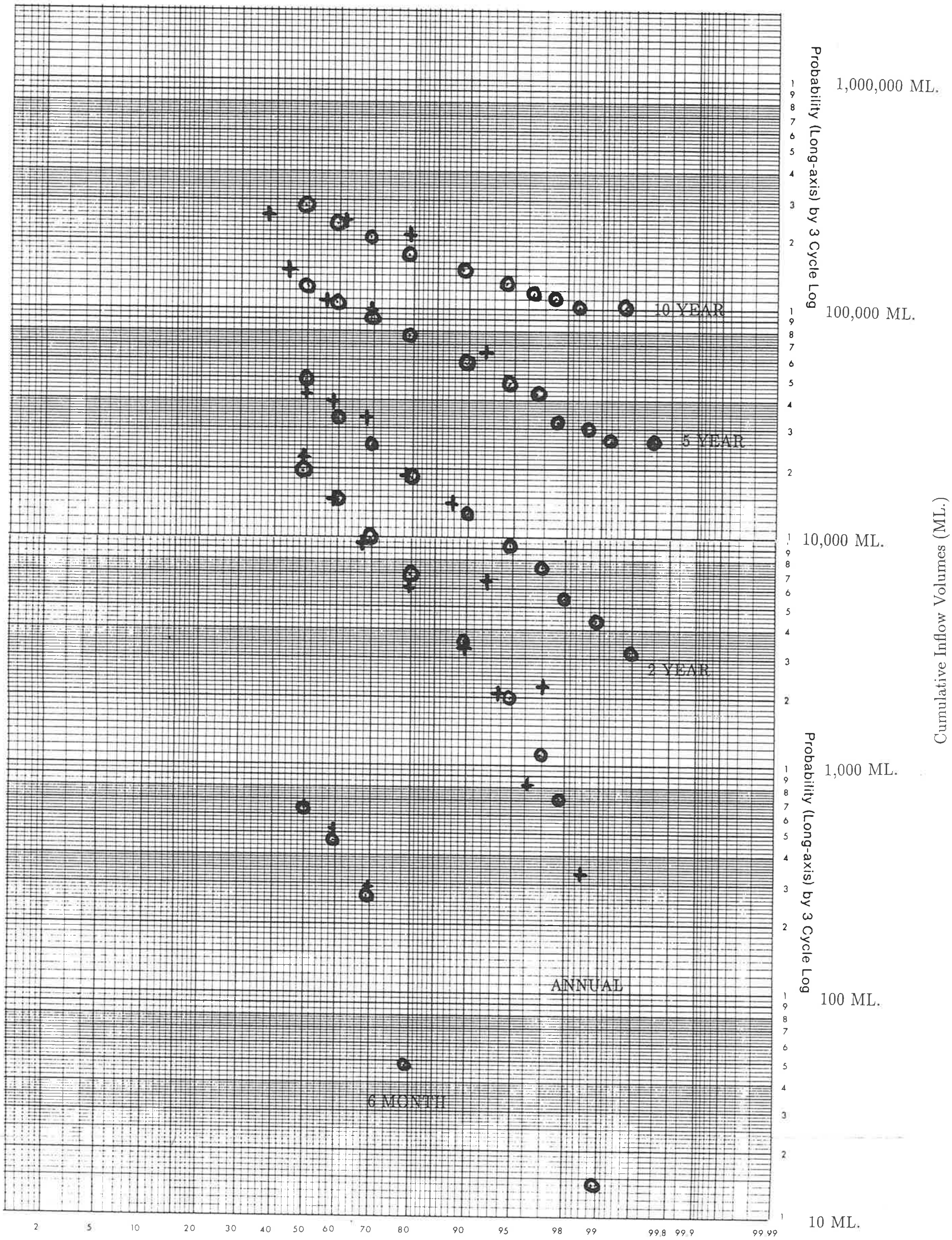


Figure 4.20: Onkaparinga Cumulative Flow Frequency Curves

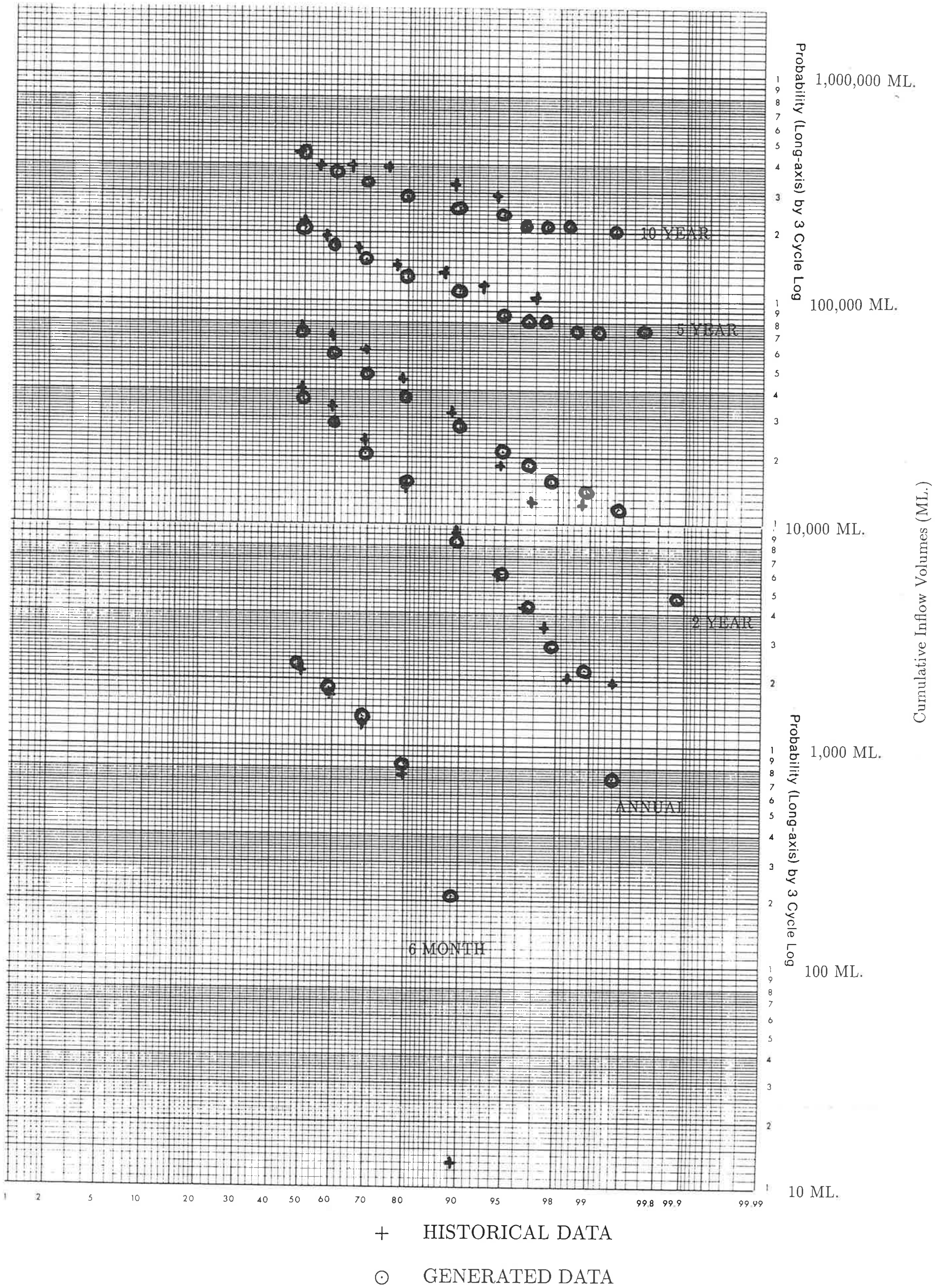


+ HISTORICAL DATA

⊙ GENERATED DATA

Exceedance Probability (%)

Figure 4.21: South Para Cumulative Flow Frequency Curves



Exceedance Probability (%)

Figure 4.22: Gorge Weir Cumulative Flow Frequency Curves

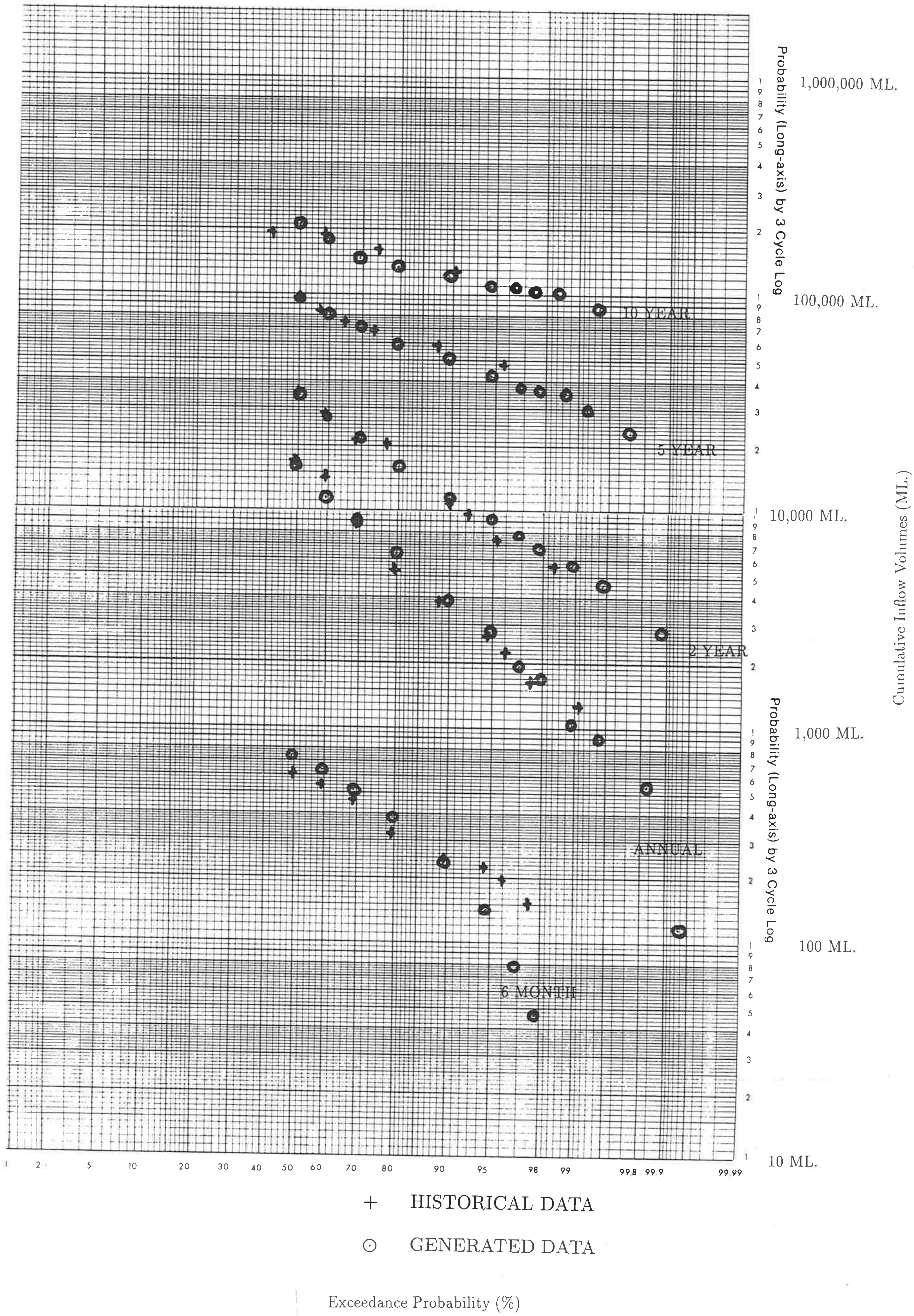


Figure 4.23: Gumeracha Weir Cumulative Flow Frequency Curves

The following observations can be drawn from the comparisons shown in these figures :

- The historical data produces five and ten year flow duration frequency curves consistently above those produced using the generated data.
- The six month, annual and two year flow duration frequency curves are fairly consistent for the historical and generated data with the notable exceptions of the South Para two year curves and the Onkaparinga annual and six month curves.

The first observation is caused because of the relatively short data sets in comparison to the flow duration event being calculated. Consider the South Para data set length of 42 years. Using the methodology described, we can obtain at best, four events to plot for the ten year flow duration curves.

As already noted, the South Para historical data set is probably the least reliable of the five records. The Onkaparinga annual and six month curves may be explained in terms of above average number of drought events occurring within the historical data.

The nature of the Adelaide system is such that because of the capacity of pumping available from the River Murray, failures are typically limited to drought events whose durations are in excess of two years. The record of synthetically generated data is observed to contain a greater proportion of drought events having periods in excess of two years than the historical data record. It is therefore likely that reliability estimates obtained for the operation of the metropolitan Adelaide system using the synthetic data record will be less than the 'true' reliability estimates. The selection of an appropriate operating rule set based on the results obtained using this data record is therefore likely to be slightly conservative.

In conclusion, the synthetic data generated for use in this study are considered suitable to examine the reliability-cost tradeoffs for the metropolitan Adelaide water supply system. It is, however, considered appropriate to examine the parameter uncertainty associated with the generation of this synthetic data.

4.4.4 Required Manipulation of the Generated Inflow Data for input to HOMA

Synthetic streamflow data and rainfall data has been generated using the model for the Adelaide catchments described in this section. This data has been generated at the five streamflow and one rainfall gauging stations. The operational simulation/optimisation model for the Adelaide headworks system (HOMA) outlined in Section 4.3 requires as input, reservoir inflows at eight locations. In order to utilise the synthetically generated streamflow data, correlation equations have been developed within this thesis to relate data at the generated sites to the required reservoir inflow sites. This work was necessary since direct reservoir inflow gauging data is not available.

4.4.4.1 South Para Subsystem

The South Para subsystem supplies the northern areas of metropolitan Adelaide together with some of the lower northern regions of the state. Details of this subsystem have been described in Section 4.2.2. A layout of the system is shown in Figure 4.5.

The synthetic data generation model has been used to generate streamflow data at the Barossa Diversion Weir. The inflow at the Barossa Diversion Weir comprises, (1) the Warren Reservoir local catchment (119 km^2), and (2) the South Para catchment downstream of Warren Reservoir (116 km^2)

as shown in Figure 4.18. It has been assumed that the areal distribution of rainfall and runoff is uniform over the whole catchment and the respective individual reservoir inflows can be assumed to be the areal proportion of the total streamflow measured at the Barossa Diversion Weir.

The respective inflow components are given by Equations 4.14 and 4.15.

$$I_{wr} = 0.5205 * I_{bar} \tag{4.14}$$

$$I_{sp} = 0.4795 * I_{bar} \tag{4.15}$$

where,

I_{wr} = Warren Reservoir monthly inflow

I_{bar} = Barossa Diversion Weir monthly flow

I_{sp} = South Para Reservoir monthly inflow

Work undertaken by Tomlinson [294] examining the streamflow data for Barossa Diversion Weir and backfigured intake data for Warren Reservoir indicates that a more accurate determination of the Warren and South Para Reservoir inflows from the Barossa Diversion Weir intake data is not possible.

In the absence of a more accurate method, Equations 4.14 and 4.15 have been used to produce the synthetic inflows for Warren and South Para Reservoirs.

It is noted that Warren Reservoir has a much smaller capacity than South Para Reservoir (4.77 GL compared with 44.8 GL). In the monthly operation of Warren Reservoir, it is common for the reservoir to fill and spill during the late winter months of most years. In contrast, South Para Reservoir has an average recurrence interval of spill of approximately 7 years under the current operating rules. During periods when Warren Reservoir is full and spilling, the distribution of inflows between the two reservoirs has no impact on the operation of the system as all inflows are taken into South Para Reservoir. The monthly historical record does not contain a situation when South Para Reservoir has been spilling while Warren Reservoir is below capacity.

4.4.4.2 Little Para Reservoir

To determine the inflows at Little Para Reservoir a coefficient developed by Tomlinson [294] has been used. Tomlinson determined a correlation coefficient using double mass curves for the two catchment yields producing Equation 4.16.

$$I_{lp} = 0.28583 * I_{bar} \tag{4.16}$$

where,

I_{lp} = Little Para Reservoir monthly inflow

In the absence of a more accurate method, Equation 4.16 has been used to produce the synthetic inflows for Little Para Reservoir.

4.4.4.3 Torrens Subsystem

There are two gauging stations that have been used on the Torrens River, Gumeracha Weir and Gorge Weir as shown in Figure 4.18. These two gauging stations do not directly measure inflow volumes into the three reservoirs in the Torrens subsystem, but must be manipulated to determine these required inflows.

If the Gumeracha Weir inflow is assumed to be taken into Millbrook Reservoir and the difference between the Gumeracha Weir inflow and the reconstructed Gorge Weir inflow is distributed on an areal basis between the three reservoirs, individual reservoir inflow estimates can be determined. Results from the application of this approach contained a considerable number of negative flow values for the areas downstream of Gumeracha Weir. After consideration of alternative adjustment methods it was decided to divide the reconstructed Gorge Weir inflow record between the three Torrens catchments based on their areal distribution.

The distribution for the three reservoirs is as follows :

Millbrook Reservoir	$(191.8 + 36.5)/(339.7)$	=	0.67206
Kangaroo Creek Reservoir	$(54.5)/(339.7)$	=	0.16044
Hope Valley Reservoir	$(56.9)/(339.7)$	=	0.16750

The respective inflow components adopted in this work are given by Equations 4.17, 4.18 and 4.19.

$$I_{mlb} = 0.67206 * I_{gw} \tag{4.17}$$

$$I_{kc} = 0.16044 * I_{gw} \tag{4.18}$$

$$I_{hov} = 0.16750 * I_{gw} \tag{4.19}$$

where,

I_{mlb} = Millbrook Reservoir monthly inflow

I_{gw} = Gorge Weir monthly flow

I_{kc} = Kangaroo Creek Reservoir monthly inflow

I_{hov} = Hope Valley Reservoir monthly inflow

The manner in which the Torrens system is operated in practice will have a mitigating effect on possible distribution errors for the respective reservoir inflow data sets. Transfers between reservoirs are undertaken on a daily basis, manipulating the three storages as water is supplied through Hope Valley Reservoir to demand, and as natural intakes from catchment runoff and pumped intakes from the Mannum-Adelaide pipeline are taken into the three reservoirs in the system. Potential flexibilities with the Torrens subsystem are highlighted in Figure 4.24. These daily adjustments are used to meet the monthly reservoir targets for the system. Given that the overall Torrens subsystem natural inflow measurements are of the highest achievable accuracy, errors in the distribution between catchments within the subsystem will not have a significant impact on the long-term operation of the headworks system.

4.4.5 Application of Generated Inflow Data

Using Equations 4.14, 4.15, 4.16, 4.17, 4.18 and 4.19, monthly synthetic streamflow data generated from the three of the five streamflow gauging sites have been translated into monthly inflows for six of the ten metropolitan Adelaide reservoirs and the remaining two inflow sites are applied directly.

It has been noted that both the historical and synthetically generated streamflow data sets contain negative streamflow values.

These negative values result from the process of reconstructing the natural streamflow records. In summer months the errors are quite high and the esti-

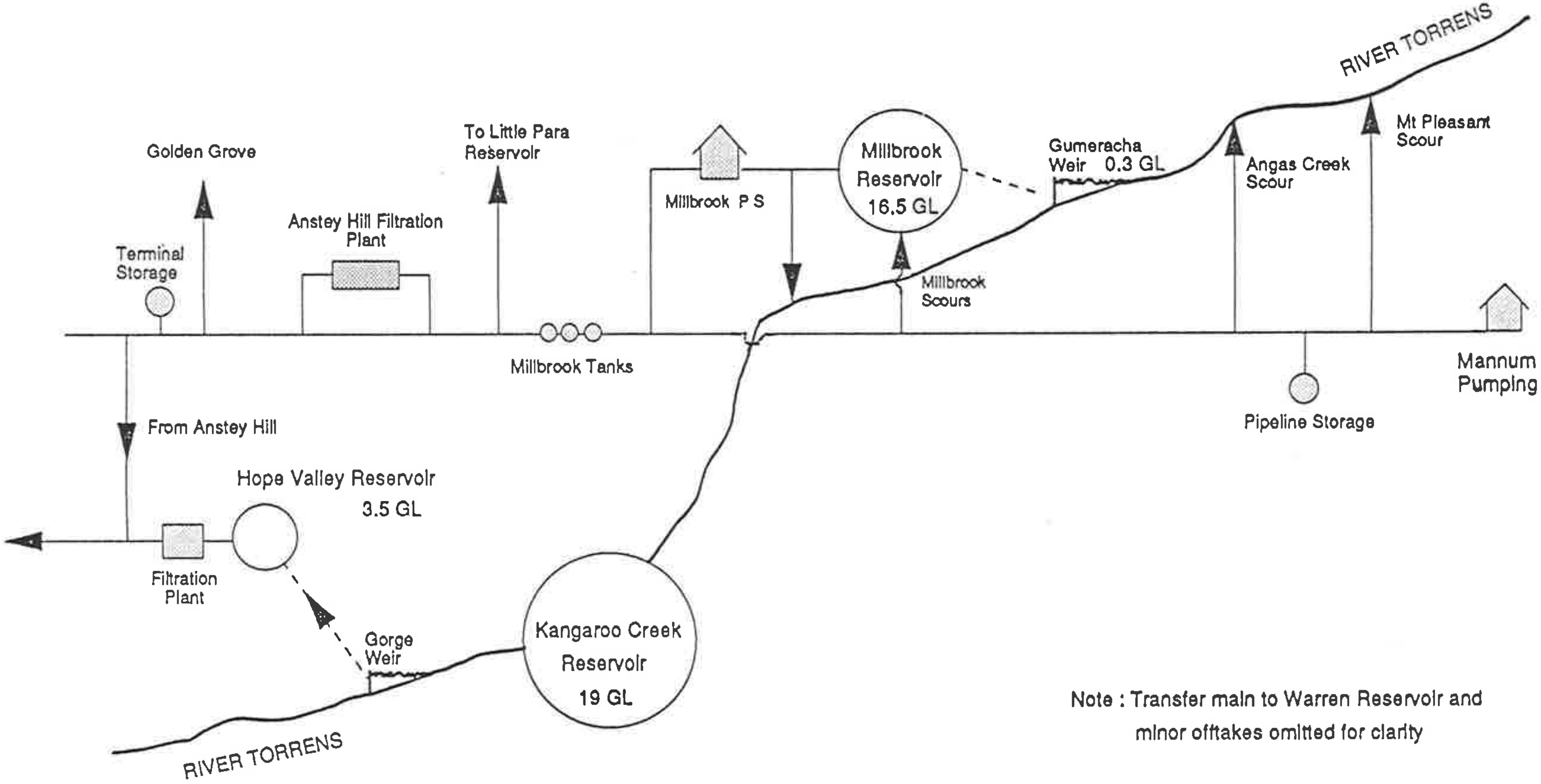


Figure 4.24: Torrens Subsystem Transfer Facilities

mated flows small. The generated synthetic streamflow data sets have therefore been adjusted to remove these negative values in the following manner.

For each generated streamflow site, the sum of inflows for a water year is calculated together with the sum of the negative inflow values for the year. The monthly inflow values are then adjusted by setting the negative values to zero and multiplying the positive values by the ratio of the sum of the absolute monthly inflows (including negative inflows) to the annual inflow (excluding negative inflows). In this way the annual generated inflow volumes are preserved and the negative inflows distributed throughout the year in proportion to the monthly inflow values.

4.4.6 Summary

In this section a first order multisite multiperiod autoregressive generation model has been described that has been used to generate synthetic streamflow data for five streamflow and one rainfall gauging sites in the Adelaide hills. Adjustments to the data generated by this model have been made to enable this data to be used as input to the HOMA model previously described. Details of these adjustments have been presented.

4.5 Synthetic Demand Data Generation

As described in Section 4.4, the synthetic inflow data generation model developed by Dandy and Baker [64] produces as part of its output, a synthetic rainfall data set at the Millbrook Reservoir rainfall gauging station. Given the availability of this data and the known correlation between rainfall and demand, a simple demand model has been developed to enable synthetic demand data to be generated for each of the demand zones in the metropolitan

Adelaide system. This section describes this demand data generation model.

4.5.1 Background to the Synthetic Demand Data Generation Model

Factors that influence urban water demand have been described in Section 3.4 and include climatic and socioeconomic parameters. When assessing the reliability of a water supply system we are generally interested in (1) the current level of reliability of the system and (2) the time until the system reliability will fall below some acceptable level requiring augmentation or modification of the system.

Within this work, the current level of reliability of the metropolitan Adelaide water supply headworks system is the primary consideration. Future trends in demands for the system have not been considered. The system demand data that is required within this research is that which best represents the statistical properties of the current demands rather than any historical or future demand patterns.

With this in mind, a number of alternative methods have been considered for the generation of synthetic demand data.

- **Method 1**

  The simplest method is to simply take the average demand data for say the last 5 years and use this as the system demand data set for each year.

$$D = M \tag{4.20}$$

where,

$$\begin{aligned} D &= \text{Demand (GL)} \\ M &= \text{Average demand over the last five years (GL)} \end{aligned}$$

- **Method 2**

A slightly more complicated method, based upon method 1 is to take the average demand data for say the last 5 years and then apply a random error term to this demand that is a function of the average monthly rainfall. As the rainfall decreases so the demand increases.

$$D_t = M_t + E(R_t) \tag{4.21}$$

where,

$$\begin{aligned} D_t &= \text{Demand during month } (t) \text{ (GL)} \\ M_t &= \text{Average demand during month } (t) \text{ (GL)} \\ E() &= \text{Random error function (GL)} \\ R_t &= \text{Average monthly rainfall during month } (t) \text{ (mm)} \end{aligned}$$

- **Method 3**

The most elaborate method is to take the monthly rainfall and develop correlation equations with the system demands. The monthly rainfall generated with the synthetic inflow can then be applied to these correlation equations to determine appropriate demand values.

$$D_t \;=\; m_t(R_t) \;+\; C_t \tag{4.22}$$

where,

D_t	=	Demand during month (t) (GL)
$m_t()$	=	Monthly demand function for month (t) (GL)
R_t	=	Average monthly rainfall during month (t) (mm)
C_t	=	Rainfall correlation constant during month (t) (GL)

4.5.1.1 Adopted Synthetic Demand Data Generation Method

The adopted demand data generation technique is to consider the historical demands and develop correlation equations between these demands and monthly rainfalls at a representative rainfall station for each of the demand locations considered.

For each of the demand zones considered, a correlation equation for each month is developed. These equations take the following form :

$$D_{jt} \;=\; m_{jt}R_t \;+\; C_{jt} \tag{4.23}$$

where,

D_{jt}	=	Demand for demand zone (j) during month (t) (GL)
m_{jt}	=	Rainfall correlation coefficient for demand zone (j)

during month (t) (GL)

R_t = Rainfall at Millbrook Reservoir during month (t) (mm)

C_{jt} = Rainfall correlation constant for demand zone (j) during month (t) (GL)

4.5.2 The Synthetic Demand Data Generation Model

The synthetic demand data generated for the metropolitan Adelaide water supply system in this research work is to be used to test the performance of the system under the current demand conditions. Future trends in demand due to changes in population or water use patterns have not been considered.

The demand data generation technique that has been used to generate synthetic demand data for the metropolitan Adelaide water supply system has been described in Section 3.4.

Demands for the water years 1984/85 to 1990/91 have been used to develop correlation equations between demands and the monthly rainfalls at Millbrook reservoir.

The following sets of demand figures have been considered :

- Demand from Lower North Demand Zone (NDWR)
- Cumulated Demand from Barossa, Little Para, Anstey Hill and Hope Valley Demand Zones (NDBR+NDLP+NDAH+NDHV)
- Mannum-Adelaide Pumping System On-line Demand (PMAO)
- Demand from Happy Valley Demand Zone (SDHV)
- Demand from Encounter Bay Demand Zone (SDEB)

- Demand from Myponga Demand Zone (SDMY)

- Murray Bridge-Onkaparinga Pumping System On-line Demand (PM-BOO)

as shown in the system schematics in Figures 4.2 and 4.4.

For each of these sets of demand figures a correlation equation has been developed for each month taking the following form :

$$D_{jt} = m_{jt}R_t + C_{jt}$$

where,

$$\begin{aligned}
D_{jt} &= \text{Demand for demand zone } (j) \text{ during month } (t) \text{ (GL)} \\
m_{jt} &= \text{Rainfall correlation coefficient for demand zone } (j) \\
&\quad\ \text{during month } (t) \text{ (GL)} \\
R_t &= \text{Rainfall at Millbrook Reservoir during month } (t) \text{ (mm)} \\
C_{jt} &= \text{Rainfall correlation constant for demand zone } (j) \\
&\quad\ \text{during month } (t) \text{ (GL)}
\end{aligned}$$

The derived demand data correlation coefficients for each of the demand sites in the northern and southern subsystems of the Adelaide headworks system are presented in Tables 4.20, 4.21, 4.22, 4.24, 4.25, 4.26 and 4.27. These coefficients have been derived in order to generate synthetic demand data consistent with the synthetic inflow data generated.

The coefficients have been derived from seven years of historical monthly demand data correlated against monthly rainfall at the Millbrook rainfall station.

4.5.2.1 Northern System

Tables 4.20, 4.21 and 4.22 contain the coefficients for the Warren demands, the overall northern system demands and the Mannum-Adelaide online demands respectively. Correlation coefficients were derived for the individual demand centres in the northern system however it was found that the 'r^2' terms for these coefficients were poor. Correlations coefficients were therefore determined for the overall northern system demands and the generated demands apportioned to the demand zones according to the average proportions for the seven year period (1984-1991) as given in Table 4.23. Because there is some flexibility downstream of the water filtration plants with potential to transfer between demand zones, the synthetic demands generated are considered satisfactory.

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	0.398	-	-	-
August	0.00000	0.372	-	-	-
September	0.00000	0.433	-	-	-
October	-0.00260	0.777	0.00210	0.144	0.234
November	-0.00665	1.164	0.00253	0.130	0.579
December	-0.00287	1.380	0.00177	0.117	0.344
January	-0.00565	1.616	0.00177	0.069	0.670
February	-0.01388	1.507	0.00534	0.129	0.574
March	-0.00376	1.274	0.00327	0.199	0.209
April	0.00000	0.848	-	-	-
May	0.00000	0.538	-	-	-
June	0.00000	0.348	-	-	-

Table 4.20: Warren System Demand (NDWR) Rainfall Correlation Coefficients

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	5.000	-	-	-
August	0.00000	5.600	0.01074	0.552	0.397
September	-0.02112	7.769	0.01415	0.928	0.308
October	-0.02826	9.530	0.02115	1.447	0.263
November	-0.02083	10.748	0.02413	1.240	0.129
December	-0.02765	12.827	0.01708	1.129	0.344
January	-0.03361	14.172	0.01291	0.563	0.575
February	-0.08368	12.714	0.03189	0.767	0.579
March	-0.04115	12.534	0.01397	0.850	0.634
April	-0.01742	8.743	0.01074	0.552	0.397
May	-0.01468	8.516	0.00901	1.264	0.347
June	0.00000	5.600	-	-	-

Table 4.21: Northern System Demand (NDBR+NDLP+NDAH+NDHV) Rainfall Correlation Coefficients

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	0.136	-	-	-
August	0.00000	0.121	-	-	-
September	0.00000	0.131	-	-	-
October	0.00000	0.140	-	-	-
November	-0.00177	0.289	0.00153	0.078	0.212
December	0.00000	0.286	-	-	-
January	0.00000	0.390	-	-	-
February	-0.00597	0.400	0.00312	0.075	0.422
March	-0.00158	0.370	0.00189	0.116	0.122
April	0.00000	0.190	-	-	-
May	0.00000	0.131	-	-	-
June	0.00000	0.110	-	-	-

Table 4.22: Mannum-Adelaide Online Demand (PMAO) Rainfall Correlation Coefficients

Month	Demand Zones			
	NDBR	NDLP	NDAH	NDHV
July	0.218	0.025	0.403	0.354
August	0.204	0.010	0.472	0.314
September	0.183	0.022	0.471	0.324
October	0.160	0.113	0.383	0.344
November	0.141	0.134	0.373	0.352
December	0.141	0.144	0.372	0.343
January	0.139	0.166	0.373	0.322
February	0.143	0.148	0.380	0.329
March	0.139	0.144	0.385	0.332
April	0.139	0.122	0.402	0.337
May	0.181	0.062	0.476	0.281
June	0.197	0.010	0.475	0.318

Table 4.23: Northern System Monthly Demand Proportions

4.5.2.2 Southern System

Tables 4.24, 4.25, 4.26 and 4.27 contain the coefficients for the Happy Valley demands, the Myponga demands, the Encounter Bay demands and the Murray Bridge-Onkaparinga online demands respectively. The coefficients have been derived based on the current operating procedures in which filtered water is supplied from Happy Valley to as large a proportion of the southern districts as possible.

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	3.507	-	-	-
August	0.00000	3.530	-	-	-
September	0.00000	3.695	-	-	-
October	-0.02263	7.230	0.00838	0.468	0.646
November	-0.03026	8.564	0.02200	1.132	0.274
December	-0.01474	8.838	0.01397	0.923	0.182
January	-0.05354	10.483	0.00210	0.918	0.564
February	-0.08812	9.348	0.00345	0.831	0.565
March	-0.04543	9.240	0.00147	0.896	0.656
April	0.00000	5.581	-	-	-
May	-0.01079	5.270	0.00239	0.336	0.803
June	0.00000	3.632	-	-	-

Table 4.24: Happy Valley Demand (SDHV) Rainfall Correlation Coefficients

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	0.170	-	-	-
August	0.00000	0.192	-	-	-
September	0.00000	0.227	-	-	-
October	0.00000	0.397	-	-	-
November	0.00000	0.595	-	-	-
December	-0.03216	2.643	0.02342	0.884	0.320
January	0.00000	1.686	-	-	-
February	0.00000	1.470	-	-	-
March	0.00000	1.538	-	-	-
April	-0.01466	1.700	0.01105	0.777	0.260
May	0.00000	0.619	-	-	-
June	0.00000	0.199	-	-	-

Table 4.25: Myponga Demand (SDMY) Rainfall Correlation Coefficients

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	0.063	-	-	-
August	0.00000	0.079	-	-	-
September	0.00000	0.097	-	-	-
October	0.00000	0.155	-	-	-
November	0.00000	0.257	-	-	-
December	0.00000	0.275	-	-	-
January	0.00000	0.340	-	-	-
February	0.00000	0.265	-	-	-
March	0.00000	0.211	-	-	-
April	0.00000	0.139	-	-	-
May	0.00000	0.094	-	-	-
June	0.00000	0.049	-	-	-

Table 4.26: Encounter Bay Demand (SDEB) Rainfall Correlation Coefficients

Month	'm'	'C' (GL)	Standard Error on 'm'	Standard Error on 'C' (GL)	'R^2'
July	0.00000	0.374	-	-	-
August	-0.00092	0.549	0.00041	0.054	0.505
September	-0.00230	0.638	0.00177	0.116	0.252
October	-0.00282	0.776	0.00185	0.126	0.319
November	-0.00234	0.859	0.00064	0.033	0.729
December	-0.00274	1.039	0.00080	0.053	0.704
January	-0.00472	1.112	0.00140	0.061	0.694
February	-0.00824	1.038	0.00454	0.109	0.397
March	-0.00299	0.979	0.00107	0.065	0.611
April	0.00000	0.580	-	-	-
May	0.00000	0.425	-	-	-
June	0.00000	0.328	-	-	-

Table 4.27: Murray Bridge Onkaparinga Online Demand (PMBOO) Rainfall Correlation Coefficients

It is acknowledged that the synthetic demand data generation model adopted within this research work is a simple one. Additional complexity has not been included within this model because the nature of the Adelaide system is such that the impact of demand variations is small compared with the impact of inflow variations and the reliability of the pumping system from the River Murray.

4.5.3 Summary

In this section a simple demand data generation model has been presented that utilises synthetic rainfall data generated as part of the synthetic inflow data generation model. Correlation equations have been developed relating zonal demands with rainfall. This demand data generation model allows the inclusion of demand variation in the assessment of reliability-cost tradeoffs for the metropolitan Adelaide water supply headworks system.

4.6 Component Reliability Assessment

In Section 3.5.1.3 of Chapter 3, a methodology has been presented for the identification of critical components of a bulk water transfer system and the estimation of the reliability parameters associated with these components. Using the technique of frequency-duration analysis, this set of component reliability information can be combined.

In this section, results from the application of this methodology to the metropolitan Adelaide headworks system are presented. Using frequency-duration analysis, these results have been combined and used as input to a Monte Carlo failure generation model to generate synthetic failure data for the metropolitan Adelaide bulk water transfer system.

4.6.1 Application of the 'Walking Party' Approach to the Metropolitan Adelaide Bulk Water Transfer System

As described in Section 3.5.1.3, there are five distinct phases involved in the application of the 'walking party' approach. These are :

1. Establishment of an assessment framework.
2. Individual interviews of experts.
3. Review of the individual interview outcomes.
4. The 'walking party' process.
5. Preparation and review of the final report.

The application of each of these phases to the metropolitan Adelaide bulk water transfer system is described in this section.

4.6.1.1 Establishment of an Assessment Framework

In preparation for a realistic assessment of critical components of the Adelaide bulk water transfer system, a number of key questions must be addressed in order to establish the framework for the assessment. These questions and the responses that have been obtained during this research work are presented below.

- What are the aims and purposes of the study ?

  The aim of this study is to identify critical components of the bulk water transfer system and to estimate mean repair times and failure frequencies for these components. The estimated repair times are to be based on the assumption that the repair will be carried out during a 'crisis' situation. In a 'crisis' situation, whatever resources are required will be made available to facilitate the recommencement of the operation of the system in as short a time as possible. In practical terms, it was suggested that participants consider the failure to have occurred in the middle of summer, when demands on the system were high and reservoir storage levels low.

The purpose of this study is to provide appropriate reliability information for the metropolitan Adelaide bulk water transfer system that can be used in the examination of reliability-cost tradeoffs for the overall metropolitan Adelaide water supply system.

- What is the extent of the study ?

The study is limited to the three major metropolitan Adelaide bulk water transfer pumping systems (Murray Bridge-Onkaparinga, Mannum-Adelaide and Swan Reach-Stockwell) and includes the Millbrook pump station. Longitudinal schematics for these three pumping systems have previously been presented within Section 4.2 in Figures 4.3, 4.7, and 4.12. The Morgan-Whyalla pumping system, which supplies townships in the northern Spencer Gulf region has not been considered. The study considers only components in these bulk water transfer systems and does not include failures of other components of system, such as reservoirs or water filtration plants. Low probability extreme events causing more widespread disruption to the overall water supply system, such as earthquakes, have also not been considered.

- What are the assumptions adopted in the study ?

In order to estimate the mean repair time of critical components in the bulk water transfer system, a number of important assumptions have been adopted. Firstly, it has been assumed that repairs of these critical components would be undertaken in a 'crisis' situation. Under these conditions, those involved with the repair of the components would have access to whatever available resources were required to ensure the repair process was completed in the minimum time possible. Cost was not to be considered as a limiting constraint in the proposed repair process required to make the system operational. Secondly, it has been assumed that short-term 'quick fix' techniques could be adopted, where necessary, to make the system operational as quickly as possible, even if this approach may not have been used in a scheduled repair of the same component. For example, temporary cabling of the power supply to pumps in a pump station (with appropriate protection systems) would be installed in the event of a major switchboard failure, until a new switchboard could be manufactured. Thirdly, it has been assumed that the current level of maintenance carried out on the components of the bulk water transfer system will be sustained. Reduction in the current level of maintenance is expected to increase the frequency of failure of components of the system estimated during this study.

- To what level of consequence are failure impacts to be considered ?

  The metropolitan Adelaide bulk water transfer system is used to supplement the metropolitan reservoir storages with water from the River Murray and to directly meet supply through the Anstey Hill water filtration plant. This transfer system is scheduled on a monthly basis using the optimisation model (HOMA) described in Section 4.3. Outages of the bulk water transfer system for durations of less than a week are unlikely to directly impact on the supply of water to consumers unless major changes to the current operating rules for the system were implemented. For this reason, the critical components of the bulk water transfer system considered in this study are those whose unplanned failure would reduce the capacity of the pumping system for a period of one week or greater.

- Who are the experts that need to be interviewed ?

The design, construction, operation and maintenance of the metropolitan Adelaide bulk water transfer system has involved a large group of experts from a diverse range of fields. As many of these experts as possible, are to be interviewed in order to gather the full range of available information.

- Who are the key participants that should form the 'walking party' ?

  The key participants to be selected to form the 'walking party' are to comprise experts from within the EWS department, ETSA and an outside contractor. A list of the participants selected to form the 'walking party' together with their particular area of expertise is presented in Appendix C.

- Is there sufficient overlap of knowledge among the key participants ?

  Inspection of the details of the participants presented in Appendix C reveals that in most areas, there is overlap of expertise amongst the participants. During the 'walking party' process, discussion highlighted that the overlap of expertise amongst the participants was satisfactory.

4.6.1.2 Individual interviews of experts

All of the experts selected to form the 'walking party' were individually interviewed in addition to a number of other experts who had been involved with the design, construction, maintenance and operation of the bulk water transfer system.

These interviews were informal in nature and questions of the following form were used during these interviews.

- Which components do you consider make the system most vulnerable to failure ?
- What would happen if one of these components failed ?

- What spares for these components are available ?
- How would you go about getting the system operational as quickly as possible ?
- How long would you estimate before the system was operational if one of these components failed ?
- Have these components failed in the past ? If so, how ?
- What effect did the failure of these components have on the performance of the system ?
- How quickly were these components repaired ?
- Who was involved in the repair of these components ?

4.6.1.3 Review of the individual interview outcomes

During the interview process it became clear that specific components were considered by many of those interviewed to be critical to the operation of the bulk water transfer system, while other components were considered less significant.

The information obtained during these individual interviews can be categorised into one of two broad groups of components :

- Pump station components
- Pipeline components

Information relating to the reliability of components within these two groups obtained during the interview process is outlined below.

Pump Station Components The key areas within each of the pump stations on the three major pumping systems has been identified as :

1. The power supply.
2. The power control systems (switchboards and cabling).
3. The pumps, pump motors and associated valve work.

Each of these areas was considered and a summary of the relevant information obtained from the appropriate experts having knowledge of components in these areas is presented below.

- The power supply.

  As a result of the interview with Jim Stott and Neil Riddings (ETSA) the following information has been obtained concerning the reliability of power supplied by ETSA to the pump stations.

  1. All pump stations in the metropolitan Adelaide bulk water transfer system have dual transformers. If one of these two transformers fails it is possible for the pump station to remain operational at a reduced load. The repair time for a transformer of the size used at these pump stations could be in excess of a week.
  2. The power supply to the Murray Bridge-Onkaparinga and Swan Reach-Stockwell pumping systems is fully separable. This means that the power supply to each pump station can be provided through two separate paths in the electricity transmission system. In the event of an ETSA transmission cable failure occurring between two pump stations, this failure can be remotely located and isolated by ETSA. Under these circumstances the power supply to the pump

station would be interrupted for a total maximum time of one to two hours. On the Mannum-Adelaide pumping system a transmission cable failure would need to be physically located and repaired by ETSA. Under these circumstances the power supply to the pump station would be interrupted for a total maximum time of eight hours.

3. The electricity tariff structure for the major pumping systems have been negotiated as 'interruptible supplies'. During periods when power demand exceeds ETSA's generation capacity, load shedding is implemented by ETSA. Under these conditions, power to the pumping stations may be interrupted for periods up to two hours.

On the basis of this information, the transformers were identified as important for further consideration in the component reliability examination of the bulk water transfer system.

As an outcome of the interviews with Lionel Rodrigues and Hanley Puller (Murray Bridge, EWS) the following information has been obtained concerning external pump station power supply cables. This information was also confirmed in interviews with John Minney, Paul Delaverdie and Jim Brendler (Headworks and Treatment Group, EWS).

High voltage cables are used to connect the EWS main circuit breaker to the pump station switchboard. At most pump stations in the system, this distance is short and replacement cables would be readily available and could be quickly installed. At the Murray Bridge no. 1 pump station, the cable connecting the EWS main circuit breaker to the pump station switchboard is approximately five hundred metres in length as shown schematically in Figure 4.25. Replacement cable of this length would not normally be held in stock and it is estimated that it could take up to two weeks to manufacture and install new cables in the event of complete failure.

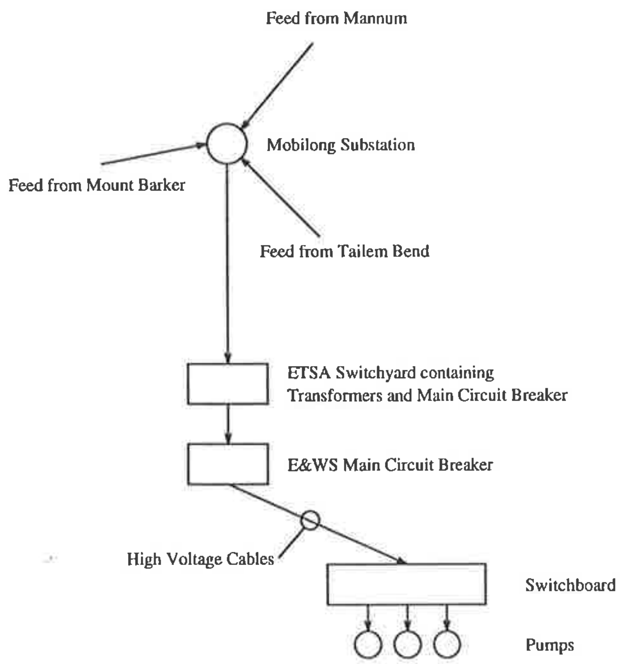


Figure 4.25: Murray Bridge-Onkaparinga No. 1 Pump Station Power Supply Schematic

On the basis of this information, further investigation of the external high voltage cables at the Murray Bridge-Onkaparinga no. 1 pump station was required in the component reliability examination of the bulk water transfer system.

- The power control systems.

Following the interview with Tony Soar (Engineering Services Branch, EWS) and Bill Hagger (ETSA) , the following information has been obtained concerning pump station internal power control systems.

1. In 1991 a failure occurred in the Millbrook pump station high voltage switchboard. The nature of the failure was such that it was pos-

sible to bypass the switchboard and allow full operation of the pump station. A new switchboard was manufactured to replace the failed switchboard. The failed switchboard in the Millbrook pump station was of the same type as those manufactured for the three Mannum-Adelaide pump stations. Those components of the failed switchboard which were undamaged are now available as spares for the three Mannum-Adelaide pump station switchboards. It is planned to replace these three pump station switchboards in the near future.

2. No specific spares are currently held in store for the Murray Bridge-Onkaparinga and Swan Reach-Stockwell pumping system switchboards.

3. A major switchboard failure in any of the pump stations would have serious implications on the bulk water transfer system. The length of time that the pump station would be 'out of action' following such an event would vary according to the nature and severity of the failure. If such an event were to occur, the required repair work would be given a high priority. Although the manufacture of a new switchboard could take up to six months, it was estimated that a temporary solution to restore operation to the pumping system could be achieved in a maximum time of two weeks.

On the basis of this information, further investigation of the pump station switchboards was considered appropriate in the component reliability examination of the bulk water transfer system.

- The pumps, pump motors and associated valve work.

During interviews with Bob Jordan (F.R. Mayfield Pty. Ltd.), John Minney, Paul Delaverdie, Jim Brendler, Lionel Rodrigues, and Hanley Puller, the following key points were noted.

1. For all pumps and pump motors in each of the pumping systems a maximum repair time was estimated of between six and eight weeks. In a 'crisis' situation these repair times could be reduced to an estimated maximum of two weeks for most of these components. Pumps and pump motor reliabilities were considered critical to the operation of the bulk water transfer system and need to be examined in greater detail.
2. The majority of the pumps and pump motors used in the Mannum-Adelaide pumping system are of the same type and size and are fully interchangeable between each of the pump stations. In the event of two pumps failing at a single pump station, an operational pump from one of the other pump stations could be relocated to replace one of the failed pumps. It is estimated that the relocation and installation of one of these pumps could be achieved in a few days. As there are three pumps at each of the three pump stations in this pumping system, failure of one pump would reduce the pumping capacity by approximately one third. An additional three pumps would then need to fail for the pumping capacity to be further reduced.
3. Pump stations 2 and 3 in the Swan Reach-Stockwell pumping system also have pumps and pump motors of the same type and size that are fully interchangeable.

On the basis of this information, further investigation of the pumps and pump motors was considered important in the component reliability examination of the bulk water transfer system.

Pipeline Components The key components of the pipeline systems are :

1. The pipes.

2. The pipeline surge protection.

3. The pipeline valves and on-line storage tanks.

4. The pipeline energy dissipator valves.

Each of these components has been considered and relevant information sought from the respective experts in these areas.

- The pipes.

  During interviews with Lionel Rodrigues and Hanley Puller, it was estimated that in the event of a major rupture in a section of pipeline, the pumping system could be out of service for at most one week. If repair was undertaken assuming a 'crisis' situation, it was estimated that these repair times could be reduced to one to two days.

  On the basis of this information, further investigation of the pipe failures was not considered necessary in the component reliability examination of the bulk water transfer system.

- The pipeline surge protection.

  During the interview with Brian Smith (Engineering Services Branch, EWS) the following information has been obtained concerning the pipeline surge protection systems.

  Pipeline surge protection is provided to prevent damage to the pipeline components by water hammer in the event of an unscheduled shutdown of pumping in the pumping system. If a component of the surge protection system for the pipeline is taken out of service, the pumping system would be operational at a reduced capacity. It is estimated that the 'worst case' scenario involving pipeline surge protection component failure would result in the pumping system being at reduced capacity for at most a week. Under 'crisis' conditions these components could be repaired in one to two days.

On the basis of this information, further investigation of the pipeline surge protection system was not considered necessary in the component reliability examination of the bulk water transfer system.

- The pipeline valves and on-line storage tanks.

As a result of the interview with Dave Kerry (Engineering Services Branch, EWS) the following information has been obtained concerning the pipeline valves and on-line storage tanks.

It is estimated that the 'worst case' scenario involving pipeline valves and on-line storage tanks would result in the pumping system being out of service for at least five days. Under 'crisis' conditions it is estimated that these components could be repaired in one to two days.

On the basis of this information, further investigation of the pipeline valves and on-line storage tanks was not considered necessary in the component reliability examination of the bulk water transfer system.

- The pipeline energy dissipator valves.

During the interview with Brian Smith the following information has been obtained concerning the pipeline energy dissipator valves.

These valves are used to dissipate hydraulic energy when water is discharged from the pipelines. The quantity of energy required to be dissipated is a function of the difference in head on the closed sections of pipeline between the summit storage and the discharge point.

The location of the energy dissipators on the Murray Bridge-Onkaparinga pumping system is shown on the longitudinal pipeline schematic presented in Figure 4.3. The energy dissipation system for this pipeline is shown in Figure 4.26 and comprises four valves ; 2 x 600 mm., 1 x 300 mm. and 1 x 100 mm. operating in parallel. If one of the 600 mm. valves were to fail, the pumping system could still be operated at full capacity using the remaining three valves. The joint probability of both 600 mm. valves being inoperable is sufficiently small to preclude the need for further examination of the energy dissipation system on the Murray Bridge-Onkaparinga pumping system.

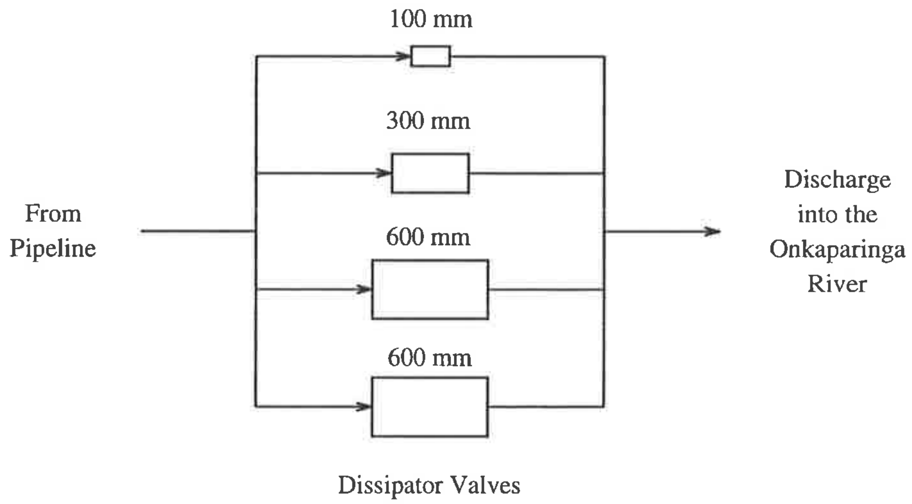


Figure 4.26: Murray Bridge-Onkaparinga Pumping System Dissipator Valve Schematic

Arrangements on the Swan Reach-Stockwell pumping system provide alternate means for discharging water into the South Para system, either directly into the Warren Reservoir or through scours into South Para River.

Release from Mannum-Adelaide pipeline into the River Torrens system can occur at the Mount Pleasant scour, the Angus Creek scour or directly into Millbrook Reservoir as previously shown in Figure 4.24. The respective capacities of these discharge points are approximately 150, 130 and 120 ML/day. The maximum daily discharge into the Torrens system is 200 ML/day. The joint probability of two scour systems being inoperable is again sufficiently small to preclude the need for further examination of the energy dissipation system on the Mannum-Adelaide pumping system.

On the basis of this information, further investigation of the pipeline energy dissipator valves was not considered necessary in the component reliability examination of the bulk water transfer system.

In summary, the following components were identified during the interview process as critical in the operation of the metropolitan Adelaide bulk water transfer system.

- Pump station high voltage switchboards
- Pumps
- Pump motors
- External high voltage cables at the Murray Bridge-Onkaparinga no. 1 pump station.
- ETSA transformers

Having identified these components as critical to the operation of the bulk water transfer system, it was possible to establish the details to be considered during the 'walking party' process. It is noted that a number of additional components were considered critical to the operation of the bulk water transfer system during the 'walking party' process that were not identified during the individual interviews.

4.6.1.4 The 'walking party' process

In October 1993, the 'walking party' process was undertaken to obtain estimates of reliability attributes of the components identified as critical to the operation of the metropolitan Adelaide bulk water transfer system . A brief summary of the skills and experience of the personnel involved in this process is presented in Appendix C.

The 'walking party' process commenced with a presentation of the background, scope, and objectives of the overall study. During the process the areas to be considered were the major pumping systems from the River Murray to metropolitan Adelaide. Early on during the process the ongoing need for regular maintenance of all system components was highlighted. As the components of the bulk water transfer system are aging, maintenance requirements will tend to increase with time until these components are replaced. Unless this maintenance is undertaken on an ongoing basis, the current level of reliability can not be maintained and the likelihood of failure of components will increase. It was assumed during the 'walking party' process that the current level of maintenance will at least be retained.

The aim of the 'walking party' process was to estimate mean repair times and failure frequencies for the components identified as critical for the operation of the system. In assessing these times it was assumed that the repairs would be carried out in 'crisis' mode; that is, whatever resources required would be made available to ensure the system was brought back into operation in as short a time as possible.

A number of additional components were identified during the 'walking party' process as critical to the operation of the bulk water transfer system. These components had not been highlighted during the interview process. The additional components identified were :

- The intake valves at Mannum
- The internal high voltage cables at the Mannum-Adelaide pump stations.
- The main circuit breakers (EWS and ETSA)
- The ETSA grid feed to the Murray Bridge-Onkaparinga no. 1 pump station

During the 'walking party' process, various sites associated with the Murray Bridge-Onkaparinga and Mannum-Adelaide pumping systems were visited together with the Millbrook pump station. During the visits, critical components were inspected and estimates for the reliability parameters of these components determined.

In addition to the parameters obtained during the 'walking party' process, useful discussion occurred particularly with regard to the electrical components of the system. This discussion highlighted that both physical and personnel resources could be made available by ETSA, in the event of a major failure of an electrical component. The provision of these resources would ensure the required repairs were undertaken in as short a time as possible.

The detailed results obtained from the 'walking party' process are presented later in this chapter, together with a summary of the reliability results obtained for the components identified as critical to the operation of the bulk water transfer system.

4.6.1.5 Preparation and review of the final report

Having completed the 'walking party' process, a draft report was prepared summarising the results obtained during the process. A copy of the draft report was sent to each of the participants seeking review and comments on the details recorded in the report. A number of useful comments and additional details were provided during this review process and included within the final report.

4.6.2 Summary of Reliability Information obtained for the Metropolitan Adelaide Bulk Water Transfer System

The three major pumping systems supplying the metropolitan Adelaide water supply system have been described in Section 4.2. Each of these systems comprises three pump stations in series that progressively lift water from the River Murray over the Mount Lofty Ranges and into reservoirs in the headworks system or directly to a water filtration plant.

A summary of the reliability information for each of these pumping systems is presented below. A more detailed description of the information obtained during the 'walking party' process is presented in Appendix D.

4.6.2.1 Murray Bridge-Onkaparinga Pumping System

The Murray Bridge-Onkaparinga pumping system has been described in Section 4.2.1 and a longitudinal schematic section of the pipeline presented in Figure 4.3.

A schematic detailing the components identified as critical to the operation of the pipeline at the Murray Bridge-Onkaparinga no. 1 pump station is shown in Figure 4.27. The critical power supply components common to each of the three pump stations in the pumping system are shown in Figure 4.28. Murray Bridge-Onkaparinga no. 2 and no. 3 pump stations have similar reliability attributes and a schematic detailing the critical components in these pump stations is shown in Figure 4.29.

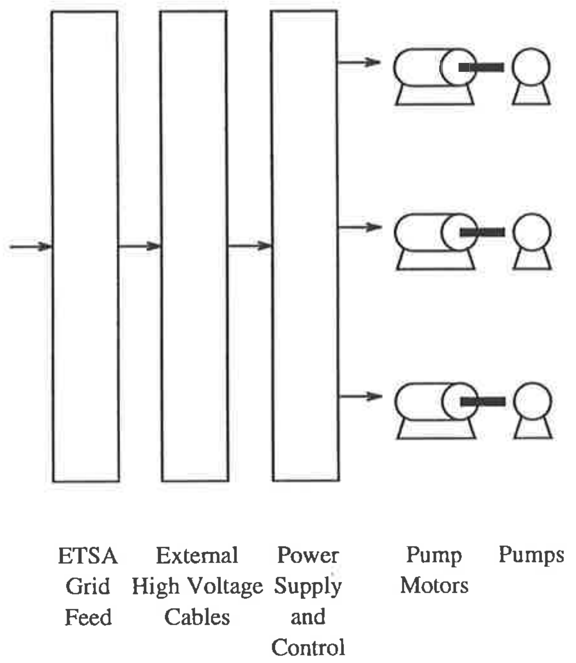


Figure 4.27: Murray Bridge-Onkaparinga Pump Station No. 1 Critical Component Schematic

Table 4.28 summarises the results obtained for the Murray Bridge-Onkaparinga pumping system used to supply the southern metropolitan Adelaide areas. The information contained in this table is also presented in Appendix D in Table D.9.

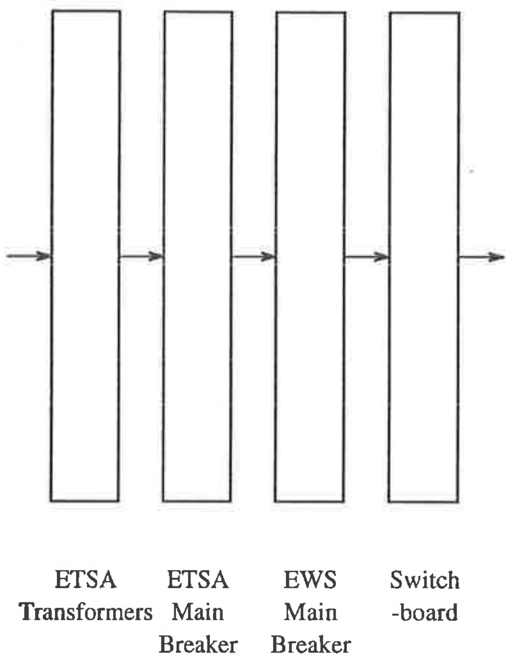


Figure 4.28: Murray Bridge-Onkaparinga Pump Stations Critical Power Supply and Control Component Schematic

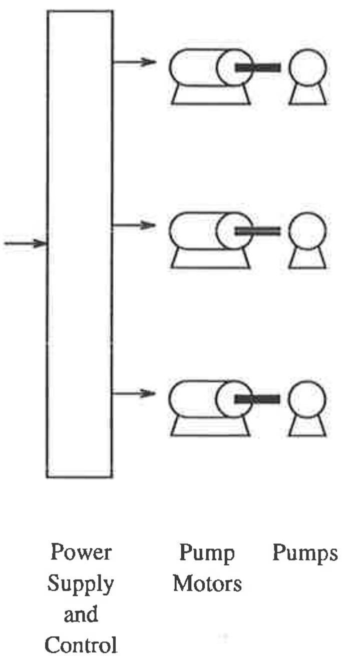


Figure 4.29: Murray Bridge-Onkaparinga Pump Station No. 2 and No. 3 Critical Component Schematic

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
MBO No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	External High Voltage Cables	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233
MBO No. 2 Pump Station	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233
MBO No. 3 Pump Station	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233

Table 4.28: Murray Bridge-Onkaparinga (MBO) Pumping System Critical Component Reliability Data

4.6.2.2 Mannum-Adelaide Pumping System

Components at the Mannum-Adelaide no. 1 pump station are typical of the components at the other two pump stations in the Mannum-Adelaide pumping system. A longitudinal pipeline schematic for the Mannum-Adelaide pumping system has been previously presented in Figure 4.12 in Section 4.2.2.

A schematic detailing the components identified as critical to the operation of the pumping system at the Mannum-Adelaide no. 1 pump station is shown in Figure 4.30. Details of the critical power supply components common to each of the three pump stations in the pumping system are presented in Figure 4.31. Mannum-Adelaide no. 2 and 3 pump stations have similar reliability attributes and a schematic detailing the critical components in these pump stations is shown in Figure 4.32. Two types of pumps and pump motors have been shown in these schematics. Type 'A' pumps are vertical centrifugal pumps while type 'B' pumps are horizontal pumps. Reliability attributes for the critical components shown in these schematics are summarised in Table 4.29. The information contained in Table 4.29 is also presented in Appendix D in Table D.10.

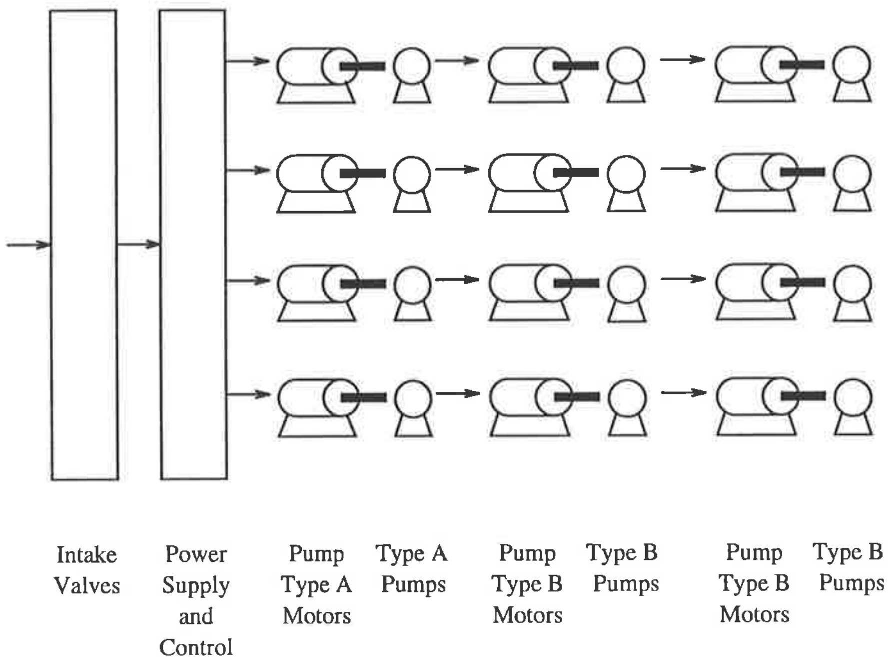


Figure 4.30: Mannum-Adelaide Pump Station No. 1 Critical Component Schematic

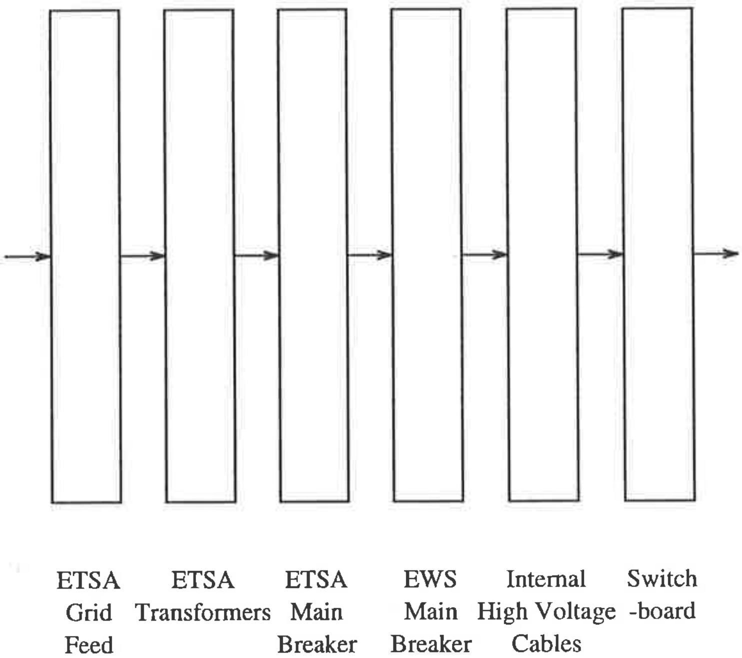


Figure 4.31: Mannum-Adelaide Pump Stations Critical Power Supply and Control Component Schematic

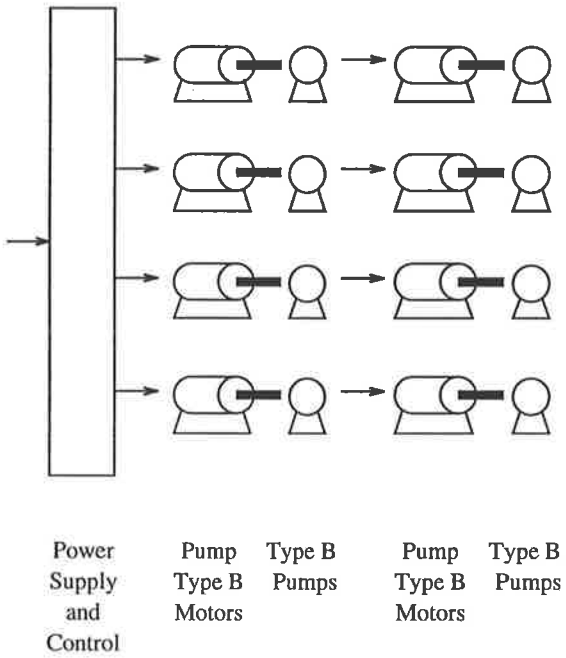


Figure 4.32: Mannum-Adelaide Pump Station No. 2 and No. 3 Critical Component Schematic

4.6.2.3 Millbrook Pump Station

A schematic detailing the components identified as critical to the operation of the Millbrook pump station is shown in Figure 4.33.

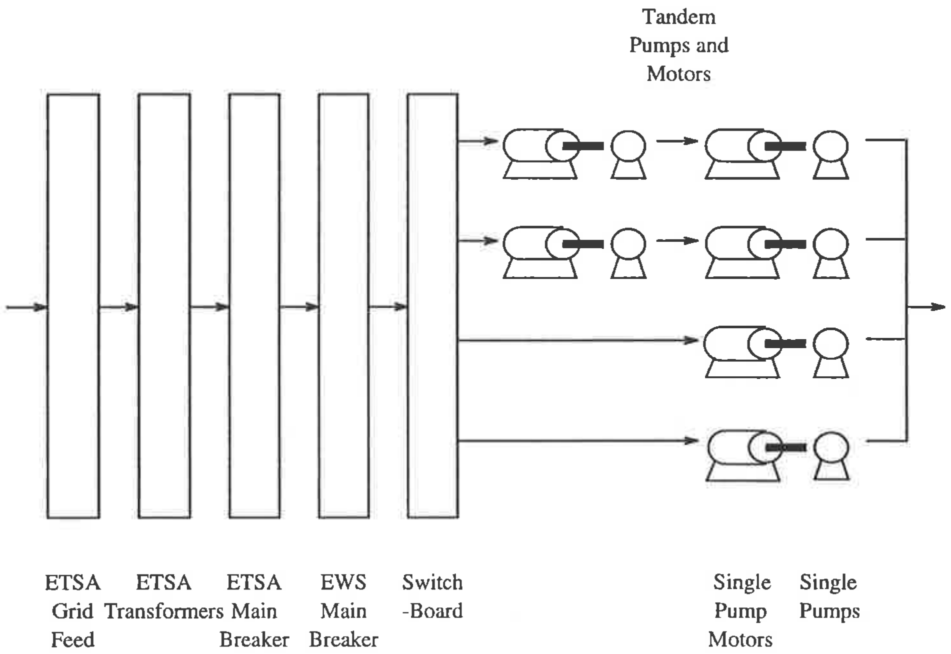


Figure 4.33: Millbrook Pump Station Critical Component Schematic

Reliability attributes for the critical components shown in this schematic are summarised in Table 4.30. The information contained in Table 4.30 is also presented in Appendix D in Table D.11.

4.6.2.4 Swan Reach-Stockwell Pumping System

During the 'walking party' process, the Swan Reach-Stockwell pumping system was not directly considered because of time limitations and the smaller impact of this pumping system on the overall reliability of the metropolitan Adelaide headworks system.

The age and form of construction of components on the Swan Reach-Stockwell

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Man-Ad. No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082
	Intake Valves	7	1 in 20	0.999041
Man-Ad. No. 2 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082
Man-Ad. No. 3 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082

Table 4.29: Mannum-Adelaide (Man-Ad) Pumping System Critical Component Reliability Data

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Millbrook Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 50	0.999616
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Switchboard	7	1 in 6000	0.999997
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082

Table 4.30: Millbrook Pump Station Critical Component Reliability Data

pumping system are similar to both the Mannum-Adelaide and Murray Bridge-Onkaparinga pumping systems. Component reliability information obtained during the 'walking party' process for these two pumping systems have been assumed for the components on the Swan Reach-Stockwell pumping system.

The ETSA grid feed, ETSA transformers, ETSA main circuit breakers, pumps and pump motors associated with the Swan Reach-Stockwell pumping system are similar to those on the Mannum-Adelaide pumping system and so these reliability data have been assumed.

The EWS main circuit breakers and switchboards associated with the Swan Reach-Stockwell pumping system are similar to those on the Murray Bridge-Onkaparinga pumping system and so these reliability data have been assumed.

Reliability attributes for the critical components assumed for the Swan Reach-Stockwell pumping system are summarised in Table 4.31. The information contained in Table 4.31 is also presented in Appendix D in Table D.12.

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Swan Reach-Stockwell No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233
Swan Reach-Stockwell No. 2 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233
Swan Reach-Stockwell No. 3 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233

Table 4.31: Swan Reach-Stockwell Pumping System Critical Component Reliability Data

4.6.3 Application of Frequency Duration Analysis

Using the results obtained from the critical component reliability assessment, a frequency duration analysis has been undertaken for the three major pumping systems and the Millbrook pump station, to produce a simplified state transition table. The results for the three pumping systems and the Millbrook pump station are presented in Tables 4.32, 4.33, 4.34, 4.35, 4.36, 4.37, 4.38 and 4.39.

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.209792	0.000000	0.002555	0.966587
2	0.149494	0.070626	0.000832	0.315474 x 10^{-1}
3	0.000624	0.142647	0.000570	0.128586 x 10^{-3}
4	0.001070	0.143751	0.000000	0.173698 x 10^{-2}
				1.000000

Table 4.32: Murray Bridge-Onkaparinga Pumping System Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (Gl/Month)
1	0.246980 x 10^{-2}	391.362244	0.404891 x 10^{3}	15.140000
2	0.225430 x 10^{-2}	13.994294	0.443596 x 10^{3}	10.100000
3	0.184157 x 10^{-4}	6.982432	0.543015 x 10^{5}	5.050000
4	0.249692 x 10^{-3}	6.956495	0.400494 x 10^{4}	0.000000

Table 4.33: Murray Bridge-Onkaparinga Pumping System Frequency Duration Analysis Results (2)

In Tables 4.32 and 4.33, state 1 represents the pumping system at full capacity, state 2 represents the system with the capacity reduced by one third, state 3 represents the system with the capacity reduced by two thirds and state 4 represents the system with zero capacity.

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.201754	0.000000	0.006900	0.923465
2	0.159017	0.068966	0.001803	0.694471×10^{-1}
3	0.214264	0.143845	0.002095	0.430009×10^{-4}
4	0.000381	0.214169	0.001906	0.765494×10^{-7}
5	0.007047	0.142780	0.000000	0.704529×10^{-2}
				1.000000

Table 4.34: Mannum-Adelaide Pumping System Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (Gl/Month)
1	0.637207×10^{-2}	144.923737	0.156935×10^{3}	10.408000
2	0.491471×10^{-2}	14.130466	0.203471×10^{3}	7.806000
3	0.627554×10^{-5}	6.852137	0.159349×10^{6}	5.204000
4	0.165404×10^{-7}	4.628037	0.604582×10^{8}	2.602000
5	0.100593×10^{-2}	7.003775	0.994107×10^{3}	0.000000

Table 4.35: Mannum-Adelaide Pumping System Frequency Duration Analysis Results (2)

In Tables 4.34 and 4.35, state 1 represents the pumping system at full capacity, state 2 represents the system with the capacity reduced by one quarter, state 3 represents the system with the capacity reduced by one half, state 4 represents the system with the capacity reduced by three quarters and state 5 represents the system with zero capacity.

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.213663	0.000000	0.001194	0.984874
2	0.145768	0.071073	0.000427	0.136756 x 10^{-1}
3	0.000225	0.142801	0.000317	0.210994 x 10^{-4}
4	0.000730	0.144575	0.000000	0.142921 x 10^{-2}
				1.000000

Table 4.36: Swan Reach-Stockwell Pumping System Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (Gl/Month)
1	0.117576 x 10^{-2}	837.646362	0.850511 x 10^{3}	2.040000
2	0.977815 x 10^{-3}	13.985927	0.102269 x 10^{4}	1.360000
3	0.301970 x 10^{-5}	6.987246	0.331159 x 10^{6}	0.680000
4	0.206628 x 10^{-3}	6.916842	0.483961 x 10^{4}	0.000000

Table 4.37: Swan Reach-Stockwell Pumping System Frequency Duration Analysis Results (2)

In Tables 4.36 and 4.37, state 1 represents the pumping system at full capacity, state 2 represents the system with the capacity reduced by one third, state 3 represents the system with the capacity reduced by two thirds and state 4 represents the system with zero capacity.

In Tables 4.38 and 4.39, state 1 represents the pumping system at full capacity, state 2 represents the system with the capacity reduced by one sixth, state 3

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1	0.285043	0.000000	0.001219	0.983557
2	0.072540	0.071389	0.001027	0.529301 x 10^{-2}
3	0.216282	0.071341	0.000835	0.106074 x 10^{-1}
4	0.144130	0.142683	0.000642	0.570453 x 10^{-4}
5	0.071970	0.142779	0.000450	0.286378 x 10^{-4}
6	0.000767	0.213976	0.000258	0.153701 x 10^{-6}
7	0.000590	0.144723	0.000000	0.456970 x 10^{-3}
				1.000000

Table 4.38: Millbrook Pump Station Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (Gl/Month)
1	0.119867 x 10^{-2}	820.539490	0.834257 x 10^{3}	7.500000
2	0.383299 x 10^{-3}	13.809096	0.260893 x 10^{4}	6.250000
3	0.765599 x 10^{-3}	13.855020	0.130617 x 10^{4}	5.000000
4	0.817604 x 10^{-5}	6.977133	0.122309 x 10^{6}	3.750000
5	0.410176 x 10^{-5}	6.981831	0.243798 x 10^{6}	2.500000
6	0.329281 x 10^{-7}	4.667779	0.303692 x 10^{8}	1.250000
7	0.661340 x 10^{-4}	6.909762	0.151208 x 10^{5}	0.000000

Table 4.39: Millbrook Pump Station Frequency Duration Analysis Results (2)

represents the system with the capacity reduced by one third, state 4 represents the system with the capacity reduced by one half and state 5 represents the system with zero capacity.

4.6.4 The Monte Carlo Failure Simulation Model

Using the results obtained from the application of the frequency-duration analysis method, random pumping system failure events have been generated for a 2000 year period using a Monte Carlo failure simulation model. Details of the generation of random failure events using this model have been described in Section 3.5.3 of Chapter 3. These generated failure events represent 2000 possible realisations of the failures occurring over the next twelve months under the current pumping system conditions. The data generated using this model contains the daily transitions between states for each of the pumping systems. Associated with each state is a pumping capacity. This daily synthetic data has been transformed to monthly pumping capacity data that can be used as direct input into the simulation/optimisation model for the metropolitan Adelaide headworks system. Summary information regarding the synthetically generated failure data is given in Tables 4.40, 4.41, 4.42, and 4.43.

State	Cumulative State Duration (Days)	Percentage Availability	Mean State Duration (Days)
1	704107	0.963627	376.72927
2	25101	0.034353	14.69614
3	110	0.000151	6.87500
4	1366	0.001869	7.42391
	730684	1.000000	

Table 4.40: Murray Bridge-Onkaparinga Pumping System Synthetically Generated Failure Statistics

Comparison of the summary statistics for the synthetically generated failure

State	Cumulative State Duration (Days)	Percentage Availability	Mean State Duration (Days)
1	672416	0.920394	149.06141
2	49816	0.068187	14.18047
3	5	0.000007	1.66667
4	0	0.000000	0.00000
5	8337	0.011412	7.05927
	730574	1.000000	

Table 4.41: Mannum-Adelaide Pumping System Synthetically Generated Failure Statistics

State	Cumulative State Duration (Days)	Percentage Availability	Mean State Duration (Days)
1	721853	0.984787	862.42891
2	10314	0.014071	14.46564
3	16	0.000022	8.00000
4	821	0.001120	6.17293
	733004	1.000000	

Table 4.42: Swan Reach-Stockwell Pumping System Synthetically Generated Failure Statistics

State	Cumulative State Duration (Days)	Percentage Availability	Mean State Duration (Days)
1	719379	0.984004	823.08810
2	3902	0.005337	13.73944
3	7368	0.010078	13.54412
4	60	0.000082	12.00000
5	4	0.000005	2.00000
6	0	0.000000	0.00000
7	360	0.000492	6.79245
	731073	1.000000	

Table 4.43: Millbrook Pump Station Synthetically Generated Failure Statistics

data contained in Tables 4.40, 4.41, 4.42, and 4.43 with the frequency duration analysis results contained in Tables 4.32, 4.33, 4.34, 4.35, 4.36, 4.37, 4.38 and 4.39 used as input to the Monte Carlo failure simulation model reveals that the statistical properties of failure data have been maintained. Slight differences in the failure statistics can be accounted for by sampling and rounding errors in the simulation model.

4.6.5 Inclusion of Failure Data within HOMA

As described in Section 4.3, the optimisation/simulation model (HOMA) that has been developed to assist in the operation of the metropolitan Adelaide headworks system uses a monthly time step. Data generated using the Monte Carlo failure simulation model described in the previous section comprises a record of states and the duration times in those states measured in days.

In order to include this synthetically generated failure data in the HOMA model it is necessary to convert this daily data into equivalent monthly data. Consider a pumping system failure occurring during a month and resulting in a reduction to 50% capacity of the pumping system capacity, for 50% of the month as shown in Figure 4.34. The monthly pumping system capacity used in HOMA at the commencement of the month will be the full capacity of the pumping system, as the failure event occurs during the month and is hence unknown. The model will select a level of pumping to be undertaken during the month according to the current and future forecast conditions for the system. This level of pumping is assumed to be undertaken uniformly during the month. The assumed impact of the pumping system failure and the associated reduced pumping system capacity, is to reduce the proposed level of pumping in proportion to the reduction in the pumping system capacity for the month. In the situation shown in Figure 4.34, the monthly capacity of the pumping system is reduced by $0.5 * 0.5 = 25\%$ and the planned pumping for the month would therefore be reduced by this amount.

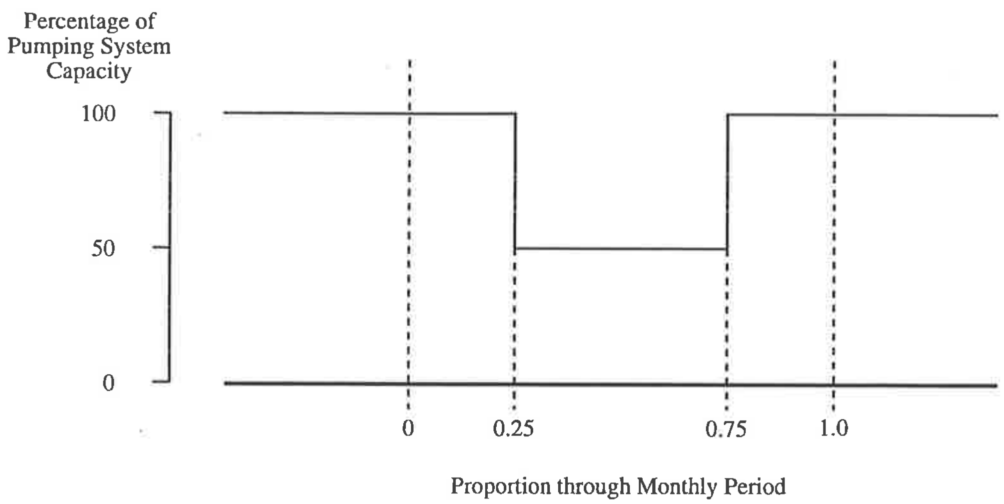


Figure 4.34: Within Month Pumping System Failure

It is recognised that this approach will yield slightly conservative results. For example if the pumping system failure occurred early during the month, then the operators of the system would have the opportunity to make up the lost pumping by pumping at a higher rate than was initially planned, to catch up on lost capacity. During critical periods for the system when pumping is being undertaken at the maximum pumping system capacity over several months, the approach adopted will realistically model the actual operation of the system.

Consider now a pumping system failure spanning two months as shown in Figure 4.35. During the first month, the failure will be 'unknown' and the approach previously described will be appropriate. For the second month, it has been assumed that the magnitude and duration of the failure will have been assessed and hence the associated reduction in capacity will be 'known'. The impact of this 'known' failure can be directly included within HOMA for determination of the planned pumping strategy. The constraint equations used in HOMA detailing the pumping system pump-cost curve and capacity can be modified to include the 'known' failure. The planned operating strategy can then take into account these 'known' failures. No adjustment to the planned pumping volumes needs to be made, as the operating strategy obtained from the model will already include the impact of the 'known' failure event.

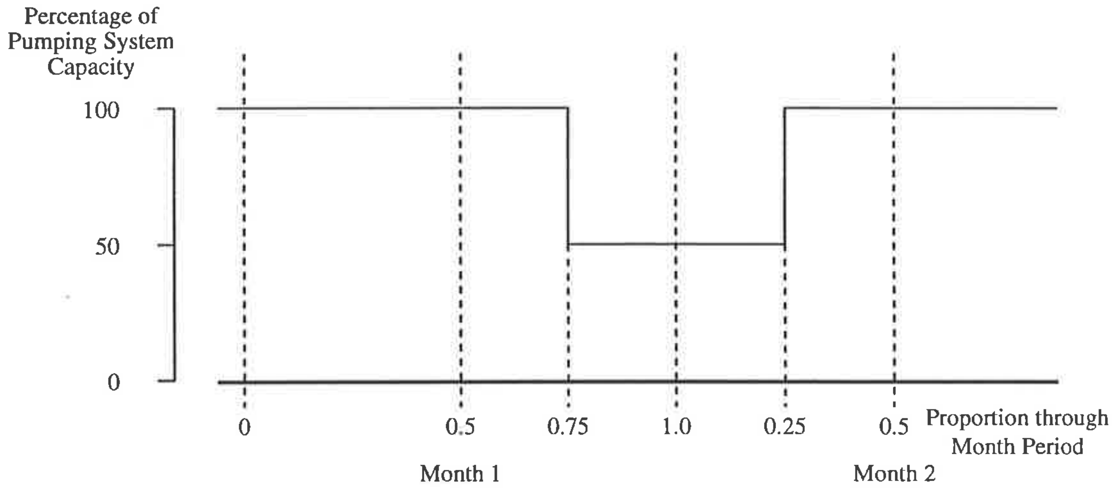


Figure 4.35: Over Month Pumping System Failure

4.6.6 Summary

In Section 3.5 of Chapter 3, a methodology for the determination of critical components of a bulk water supply transfer system and the assessment of realistic failure statistics for these components has been presented. In this section, a description has been presented of the application of this methodology to the metropolitan Adelaide water supply headworks system. Using frequency-duration analysis, data obtained from the application of this methodology has been combined. Results from the frequency-duration analysis have been used as input to a Monte Carlo failure simulation model for the generation of synthetic failure data for the metropolitan Adelaide system. The section concludes with a description of the inclusion of this failure data within the optimisation model (HOMA) used to assist in the operation of the Adelaide water supply headworks system. Using data and techniques described in this section, reliability-cost tradeoffs for the metropolitan Adelaide headworks system can be examined, including the effect of bulk water supply system failures.

4.7 Metropolitan Adelaide Water Restrictions

The need to apply water restrictions to the metropolitan Adelaide water supply system has not arisen since the construction of the Murray Bridge-Onkaparinga pumping system in 1973. In the event that the imposition of water restrictions became necessary for the Adelaide system, these restrictions would be targeted at outdoor domestic water use.

In this section, an assessment of the economic costs associated with the imposition of outdoor water use restrictions on the Adelaide headworks system has been described using the methodology presented in Section 3.6 of Chapter 3.

4.7.1 Economic Costs associated with Outdoor Water Restrictions for the Metropolitan Adelaide Water Supply System

Various studies in the United States have been undertaken to estimate the long run price elasticity of demand for outdoor water use. The climate and pattern of outdoor water use in Adelaide is most similar to areas in the western United States. Values obtained for the western United States include by Howe and Lineweaver [148] -0.73, Ben-Zvi [17] -0.82, Morris and Jones [213] -0.73 and Wilson [319] -0.5. A study of the Perth water supply system [204] found a price elasticity of -0.31 for outdoor water use. This lower value results largely because most Perth water users have the option of installing private bores, an option not available to Adelaide water users.

Work undertaken by Dandy [59] studying the factors affecting the residential water consumption in metropolitan Adelaide, estimated the long run price elasticity of demand for total water use was to equal -0.4. Given that in-house water use with a lower level of price elasticity, will account for a portion of the total water use, a value of -0.7 has been adopted in this thesis for the long run price elasticity of demand for outdoor water use in metropolitan Adelaide.

A nominal annual water allowance of 136 Kl is given to all metropolitan Adelaide domestic consumers. A fixed charge is made for this water allowance regardless of whether the full allowance is consumed. In 1994, this charge was set at \$120/annum and was based on a unit price of 0.88 \$/Kl for the full 136 Kl. Water consumed in excess of this water allowance is charged at the unit price of 0.88 \$/Kl. The vast majority of consumers in metropolitan Adelaide use in excess of their 136 Kl/annum water allowance. Details of the domestic water pricing structure for metropolitan Adelaide are shown in Figure 4.36.

In Section 3.6 of Chapter 3, details were given for the economic loss associated with the implementation of outdoor water use restrictions. If individual households are assumed to have demand functions with constant elasticity (not equal to -1), then the economic loss is given by Equation 4.24.

$$\begin{aligned} L \;=\; & p\, Q_o \left\{ \left(\frac{\bar{\varepsilon}}{1+\bar{\varepsilon}}\right) [1 - (1-\bar{r})^{(1+\bar{\varepsilon})/\bar{\varepsilon}}] \right. \\ & \left. - \frac{1}{2\bar{\varepsilon}} (1-\bar{r})^{(1-\bar{\varepsilon})/\bar{\varepsilon}}\, S_r^2 \;+\; (1-\bar{r})^{1/\bar{\varepsilon}}\, \rho_{qr}\, V_q\, S_r \right\} \end{aligned} \tag{4.24}$$

where,

L = Economic loss (\$M/month).

p = Water price = \$0.88/Kl = \$M0.88/Gl

Q_o = Total unrestricted outdoor water demand at \$0.88/Kl (Kl/month).

$\overline{r}$ = Mean fractional reduction in monthly household outdoor water demand due to restriction imposition (%).

S_r = Standard deviation of fractional reduction in individual household water demands due to restriction imposition.

V_q = Coefficient of variation of individual household outdoor water demand functions.

$\overline{\varepsilon}$ = Mean price elasticity of demand for outdoor water use for all households (-0.7).

ρ_{qr} = Cross correlation coefficient between individual outdoor household water consumptions and fractional reduction in individual household water demands due to restriction imposition.

When the standard deviation of fractional reduction in individual household water demands due to restriction imposition (S_r) is small, Equation 4.24 reduces to Equation 4.25.

$$L = p\,Q_o \left(\frac{\overline{\varepsilon}}{1+\overline{\varepsilon}}\right)\left[1-(1-\overline{r})^{(1+\overline{\varepsilon})/\overline{\varepsilon}}\right] \qquad (4.25)$$

If an economic loss factor F is defined by Equation 4.26, substituting appropriate values for the metropolitan Adelaide water supply system produces Equation 4.27.

$$F = \left(\frac{\overline{\varepsilon}}{1+\overline{\varepsilon}}\right)\left[1-(1-\overline{r})^{(1+\overline{\varepsilon})/\overline{\varepsilon}}\right] \qquad (4.26)$$

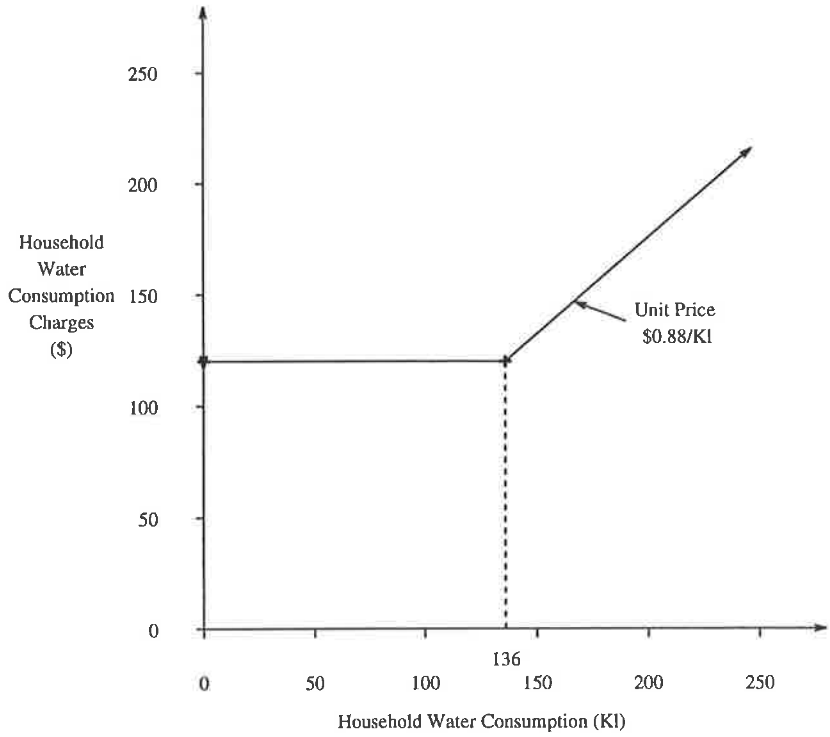


Figure 4.36: Metropolitan Adelaide Domestic Water Pricing Structure (1994)

$$F = -2.333[1 - (1 - \overline{r})^{(-0.429)}] \tag{4.27}$$

The total economic loss factor (F) associated with the imposition of restrictions comprises two components : a producer revenue loss factor (F_p) and a consumer surplus loss factor (F_c). The producer revenue loss factor is equal to the mean fractional reduction in monthly household outdoor water demand due to the imposition of restrictions ($\overline{r}$). The consumer surplus loss factor will then be given by Equation 4.28

$$F_c = F - F_p \tag{4.28}$$

The behaviour of the producer surplus loss factor (F_p) and the consumer surplus loss factor (F_c), in response to the imposition of outdoor water use restrictions can be determined for the metropolitan Adelaide water supply system as shown in Table 4.44. The information presented in Table 4.44 is shown graphically in Figure 4.37.

Reduction in Outdoor Consumption ($\bar{r}$) (%)	Total Economic Loss Factor (F)	Producer Revenue Loss Factor (F_p)	Consumer Surplus Loss Factor (F_c)
0	0.000000	0.00	0.000000
5	0.051840	0.05	0.001840
10	0.107732	0.10	0.007732
15	0.168463	0.15	0.018463
20	0.234374	0.20	0.034374
25	0.306450	0.25	0.056450
30	0.385740	0.30	0.085740
35	0.473564	0.35	0.123564
40	0.571610	0.40	0.171610
45	0.682082	0.45	0.232082
50	0.807918	0.50	0.307918
55	0.953144	0.55	0.403144
60	1.123456	0.60	0.523456
65	1.327240	0.65	0.677240
70	1.577477	0.70	0.877477
75	1.893181	0.75	1.143181
80	2.320427	0.80	1.520427

Table 4.44: Producer and Consumer Economic Loss Components of Imposed Outdoor Water Use Restrictions

As the level of restrictions increases, the proportion of the consumer surplus loss factor to total economic loss factor increases dramatically. The nature of this consumer surplus loss response suggests that less stringent water restrictions imposed early in a period of water shortage, would involve lower economic losses than the equivalent reduction in water use resulting from more stringent water restrictions imposed later.

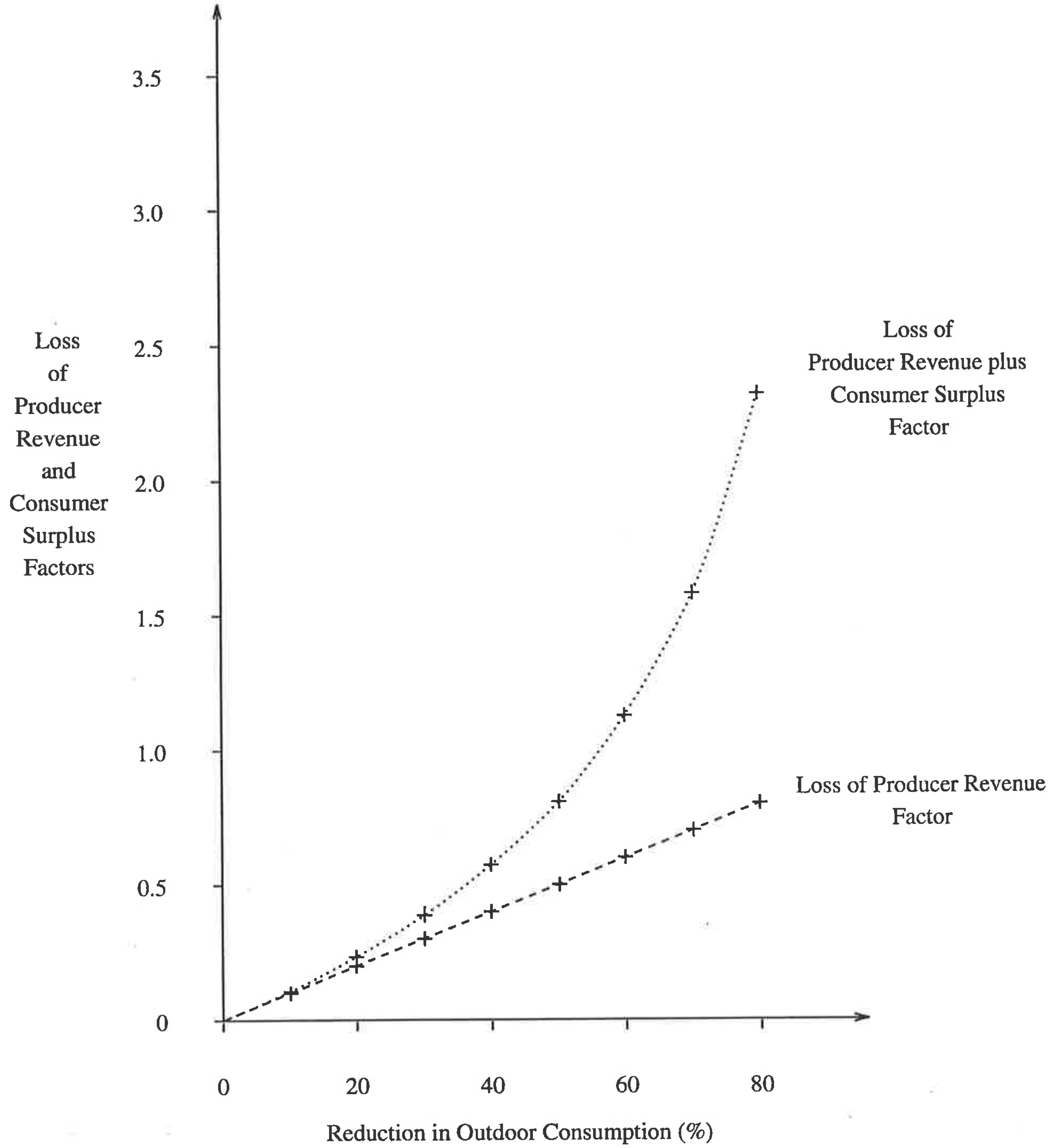


Figure 4.37: Producer and Consumer Economic Loss Components of Imposed Outdoor Water Use Restrictions

If the price and long run price elasticity of demand data for metropolitan Adelaide is entered into Equation 4.25 then Equation 4.29 can obtained describing the economic loss for metropolitan Adelaide resulting from the imposition of outdoor water use restrictions.

$$L = 0.88 \; Q_o \; F \tag{4.29}$$

If the reduction in total outdoor water demand is defined by Q_r, then Q_r is given by Equation 4.30.

$$Q_{\overline{r}} = \overline{r} \; Q_o \tag{4.30}$$

Substituting Equation 4.30 in Equation 4.29 yields Equation 4.31.

$$L = 0.88 \left(\frac{Q_{\overline{r}}}{\overline{r}} \right) F \tag{4.31}$$

Rearranging Equation 4.31, the economic loss per unit volume of reduced water demand is given by Equation 4.32.

$$\left(\frac{L}{Q_{\overline{r}}} \right) = 0.88 \left(\frac{F}{\overline{r}} \right) \tag{4.32}$$

Equation 4.32 will be used to consider a set of proposed water restrictions policies for the metropolitan Adelaide water supply system and the impact of these policies on the reliability-cost tradeoffs for the operation of the system.

4.7.2 Water Restrictions Policies and Trigger Storage Levels

Most water supply authorities follow a multi-stage restriction policy where successive stages are implemented sequentially as the risks of undesirable events increase. Moreau [211] described a risk based model for the selection of a multi-stage restriction policy. This model considers systems where the intakes to reservoirs are from catchment runoff only.

A restriction policy typically comprise three components :

1. Imposition trigger conditions

   The imposition trigger conditions are the water shortage conditions which apply when it is considered necessary to implement a restriction policy. These imposition trigger conditions typically refer to the levels of storage within the reservoirs of the headworks system that 'trigger' the imposition of the restriction policy.

2. Restriction classes

   As the water shortage conditions worsen, the severity of water restrictions will be increased to maximise the time until the system completely runs out of water. This leads to the need for a set of restriction classes associated with each of the imposition trigger conditions.

3. Relaxation trigger conditions

   The relaxation trigger conditions are those when it is considered safe to relax the level of water restrictions that have been imposed to reduce system demand. These trigger conditions again typically relate to the levels of storage within the reservoirs in the headworks system. As the water shortage conditions improve, the level of water restrictions applied to the system can be progressively reduced.

Having selected a set of imposition and relaxation trigger conditions and corresponding restriction classes, it is possible using synthetic data generated for inflows and demands, to examine the effects of this restriction policy on the operation of the headworks system.

Currently there is no proposed restriction policy in place for the Adelaide headworks system. In the event of a particularly dry year, as in the 1982/83 water year, additional pumping was carried out for both the southern and northern systems in order to provide additional security of supply above the target storage levels. In hindsight, this additional pumping was unnecessary and was carried out as a result of uncertainty regarding the real capacity and reliability of the system.

Details of the imposition and relaxation trigger storage levels used in the examination of the southern Adelaide headworks system are presented in Section 5.4 of Chapter 5.

4.7.3 Proposed Restrictions Classes for the Adelaide System

Duncan and Kesari [77] described water demand in terms of restrictable and unrestrictable components. Restrictable demand is the demand which can be removed by imposition of the most severe restrictions proposed for the system. For a water supply system, base demand can be defined as the level obtained by joining a straight line between the mean winter demand for successive years. If it is assumed that indoor and industrial water use remain relatively constant throughout the year and outdoor domestic water use in winter is minimal, then the indoor and industrial use can be assumed equal to this base demand. The outdoor water use or seasonal demand for a given month will then be equal to the component of demand for the month above the base demand level. It has been observed that the maximum monthly restrictable demand is approximately equal to the monthly seasonal demand plus a small percentage of the base demand (Duncan and Kesari [77]).

Using the record of demands from individual demand zones in the metropolitan Adelaide system for the last five years, a base level of demand has been determined for each of the demand zones in the southern system. These base demand levels are presented in Appendix B in Table B.3.

If a restriction policy is defined in terms of the percentage reduction in outdoor water use, then the economic costs associated with this restriction policy can also be determined using Equation 4.29. Since the proportion of metropolitan water consumption used for outdoor purposes will vary from month to month, the impact of the restriction policy on the total reduction in demand will vary according to the month of imposition.

Duncan and Kesari [77] considered the effect of the imposition of restrictions on the Melbourne metropolitan water supply system for a number of water shortage events. The form of restrictions and the effect of their imposition are summarised in Table 4.45.

Restriction Level	Restrictable Demand Removed (%)
None	0
Low level restriction	20-30
No fixed sprinklers, hand hose any time	50
Hand hose 2 hours/day, pool filling and car washing restricted	80
Generally buckets only, outside the house	95
All traditional restrictions	100

Table 4.45: Effect of Restrictions on the Melbourne System

Using the results obtained for the effect of restrictions on the Melbourne system as a guide, the restriction classes contained in Table 4.46 are proposed for the Adelaide system.

Restriction Class	Form of Restriction	Estimated Reduction in Outdoor Water Use (%)
Class 1	Sprinklers banned 7 a.m. to 8 p.m.	20
Class 2	No fixed sprinklers, hand hose any time	50
Class 3	Hand hose 2 hours/day, pool filling and car washing restricted	80

Table 4.46: Proposed Restriction Classes for the Adelaide System

4.8 Summary

In this chapter, details required for the application of the methodology for the assessment of reliability-cost tradeoffs to the metropolitan Adelaide water supply headworks system have been presented. The chapter commences with a description of the metropolitan Adelaide water supply headworks system and the historical and current operating rules that are used to operate it. An optimisation model (HOMA), currently used by the EWS to assist in the operation of the system is then described. Details of synthetic inflow and demand data generation models developed for the Adelaide system are presented. A methodology for the determination of critical components of the bulk water supply transfer system described in Section 3.5 of Chapter 3 has been applied to the metropolitan Adelaide system and realistic values for the failure statistics for these components have been presented. Using frequency-duration analysis, these data have been combined and used as input to a Monte Carlo failure simulation model for the generation of synthetic failure data. This synthetic failure data has been appropriately modified for inclusion within HOMA. The chapter concludes with a description of the assessment of the economic costs associated with the imposition of outdoor water use restrictions on the Adelaide headworks system.

Chapter 5

Results and Discussion

5.1 Introduction

This chapter describes results obtained from the application of the simulation methodology presented in Chapter 3 to the Adelaide water supply headworks system. This chapter is structured around three main sections. In the first of these sections, reliability-cost tradeoffs for the Adelaide system are examined taking into account hydrologic variability. In the second section, reliability-cost tradeoffs are considered taking into account the bulk water transfer system reliability in addition to the hydrologic variability. In the third section, reliability-cost tradeoffs are examined including the bulk water transfer system reliability, hydrologic variability and the effects of an outdoor water use restriction policy. The chapter concludes with a summary of the results presented.

5.2 Hydrologic Reliability Assessment

This section considers the hydrologic factors affecting the reliability-cost tradeoffs for the metropolitan Adelaide water supply system. Using synthetic data generated for reservoir inflows and system demands, the southern and northern components of the system are examined using the simulation/optimisation model HOMA.

A range of operating rules are examined for both systems and their performance assessed. It has been assumed in this section that the bulk water transfer system operates with perfect reliability in both the southern and northern system. That is, pumping from the River Murray to the headworks system and pumping at Millbrook pump station can be undertaken up to the capacity of the pumping system as considered necessary for the satisfactory operation of the system. In this section, no water restriction policy has been applied in the simulation of the system.

The synthetic inflow data generation model used to generate monthly data for the five streamflow sites involves 1080 parameters comprising mean, variance and a 'shifting parameter' for each of the sites for each month plus all the coefficients in the $[A]$ and $[B]$ matrices. Although a rigorous examination of the effect of the uncertainty of each of these parameters would be ideal, this is considered not practical. As an alternative, a sensitivity analysis is presented for those parameters identified as having the greatest impact on the system reliability.

5.2.1 Southern System

In this section, a range of operating rules for the southern system are examined. By varying the operating rules, reliability-cost tradeoffs can be considered in the operation of this system.

In these comparisons, the physical minimum operating levels at which failure is assumed to occur in the southern system reservoirs is given in Table 5.1. The summer physical minimum operating level for Happy Valley Reservoir is higher than the winter level because of the configuration of the intake system into the Happy Valley Water Filtration Plant. During summer, the higher demands from the water filtration plant require the use of additional intake pumps that are located at this higher level.

Reservoir	Physical Minimum Operating Level (ML)
Mount Bold	100
Happy Valley	4200 (Winter) 7900 (Summer)
Myponga	4600

Table 5.1: Southern System Reservoir Physical Minimum Operating Levels

In the case of both the Happy Valley and Myponga Reservoirs, it is possible to draw the reservoir below these physical minimum operating levels in a crisis situation with the installation of temporary pumps and associated pipework. In the simulation of the southern system it has been assumed that short-term draw-down below these levels may occur in order to meet demand. When the reservoir level is drawn down below these levels, a 'failure' has occurred, however full demand requirements are assumed to be satisfied.

5.2.1.1 Southern System - Inflow Exceedance Comparison

As described in section 4.2.4.2 of Chapter 4, a forecast set of inflow and demand volumes are assumed at the commencement of the water year in the preparation of the pumping program for the coming year. As the year progresses, these forecast inflows and demands are updated with 'actual' inflow and demand volumes, and the pumping program for the remainder of the year modified as necessary.

The following method has been adopted by the EWS for the determination of the inflow forecast data sets. The available monthly historical inflow data sets for each month for each reservoir have been sorted into ascending order. Inflow exceedance data sets have then been obtained from these sorted lists of historical inflow sets. For example a 70% inflow exceedance set is the set of 12 monthly inflow volumes that is exceeded by 70% of the recorded historical data set. The inflow exceedance data sets determined by the EWS have been used in this work.

In this section a comparison is made for the southern system using a range of forecast inflows sets having 90% to 10% inflow exceedance values. Details of these inflow exceedance data sets for the southern system are given in Chapter 4 in Tables 4.3 and 4.4. Additional operating rules used in this comparison include the demand forecast set presented in Table 4.11 and target storage levels for the reservoirs in the system comprising the nominal minimum reservoir operating levels given in Table B.4, plus '8 weeks demand' given in Table B.5. Other details used in the HOMA simulation model are given in Appendix B.

A synthetic data record of inflow and demand of 2000 years duration has been used to examine the long term operating and failure behaviour of the system using this range of inflow forecast data sets.

The comparative pumping costs for the whole of the southern system obtained from these model runs are given in Tables 5.2, 5.3 and 5.4. The corresponding failure occurrences in the Myponga Reservoir are shown in Tables 5.5, 5.6 and 5.7. The number of months (Mths.) during which a failure occurred is given in these tables together with the number of failure events (Evts.). The failure events are defined as the consecutive period of months during which failure of a reservoir occurred. All of the observed failures were associated with Myponga Reservoir.

Water Year Range	Pumping Electricity Cost ($M) 90% Exceedance Inflow Forecasts	80% Exceedance Inflow Forecasts	70% Exceedance Inflow Forecasts
1 → 100	176.935	171.449	166.456
101 → 200	187.326	180.655	174.385
201 → 300	210.295	205.609	201.875
301 → 400	189.464	182.917	176.086
401 → 500	203.616	197.511	192.148
501 → 600	178.287	171.681	166.308
601 → 700	207.678	201.372	196.352
701 → 800	206.662	199.541	192.352
801 → 900	184.129	178.584	173.900
901 → 1000	191.422	185.783	180.727
1001 → 1100	220.391	215.050	209.796
1101 → 1200	213.282	207.401	202.540
1201 → 1300	186.560	181.212	175.780
1301 → 1400	197.674	190.894	186.090
1401 → 1500	188.660	182.361	176.934
1501 → 1600	201.610	196.093	190.697
1601 → 1700	192.379	186.780	181.725
1701 → 1800	200.278	195.681	191.297
1801 → 1900	208.173	202.768	197.244
1901 → 2000	177.253	172.030	166.783
Total	3921.984	3805.372	3699.476
Average Annual Cost	1.961	1.903	1.850

Table 5.2: Southern System Pumping Costs for Various Inflow Exceedance Levels - (1)

Water Year Range	Pumping Electricity Cost ($M)		
	60% Exceedance Inflow Forecasts	50% Exceedance Inflow Forecasts	40% Exceedance Inflow Forecasts
1 → 100	164.618	164.037	168.614
101 → 200	170.344	170.000	178.982
201 → 300	200.205	201.466	206.077
301 → 400	172.677	171.522	175.134
401 → 500	189.402	187.800	190.595
501 → 600	164.109	164.415	168.394
601 → 700	193.232	191.727	194.035
701 → 800	188.101	186.713	188.919
801 → 900	171.770	171.356	176.032
901 → 1000	177.464	176.292	179.292
1001 → 1100	207.609	207.626	211.763
1101 → 1200	198.777	197.613	200.025
1201 → 1300	173.026	172.423	175.874
1301 → 1400	184.043	183.553	187.242
1401 → 1500	174.130	174.359	178.936
1501 → 1600	187.867	187.822	191.311
1601 → 1700	179.637	178.529	181.192
1701 → 1800	189.506	188.806	193.344
1801 → 1900	195.743	194.335	197.480
1901 → 2000	165.255	165.040	169.191
Total	3647.515	3635.434	3706.432
Average Annual Cost	1.824	1.818	1.853

Table 5.3: Southern System Pumping Costs for Various Inflow Exceedance Levels - (2)

Water Year Range	Pumping Electricity Cost ($M)		
	30% Exceedance Inflow Forecasts	20% Exceedance Inflow Forecasts	10% Exceedance Inflow Forecasts
1 → 100	176.562	183.479	193.181
101 → 200	179.976	185.417	195.416
201 → 300	213.024	219.557	229.376
301 → 400	181.304	187.445	197.639
401 → 500	198.486	204.464	214.251
501 → 600	176.754	184.758	197.121
601 → 700	200.068	205.971	215.611
701 → 800	195.194	199.852	207.573
801 → 900	183.364	190.294	200.407
901 → 1000	186.006	190.831	198.946
1001 → 1100	219.217	225.339	234.531
1101 → 1200	205.761	210.207	217.843
1201 → 1300	181.998	187.668	196.919
1301 → 1400	194.204	199.266	209.044
1401 → 1500	187.680	195.371	207.972
1501 → 1600	198.469	204.368	214.106
1601 → 1700	186.876	193.105	202.860
1701 → 1800	199.582	205.021	214.658
1801 → 1900	204.992	211.145	220.210
1901 → 2000	176.518	183.957	194.638
Total	3846.035	3967.515	4162.302
Average Annual Cost	1.923	1.984	2.081

Table 5.4: Southern System Pumping Costs for Various Inflow Exceedance Levels - (3)

Water Year Range	Myponga Failure Occurrence					
	90% Exceedance Inflow Forecasts		80% Exceedance Inflow Forecasts		70% Exceedance Inflow Forecasts	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0
101 → 200	0	0	0	0	0	0
201 → 300	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0
601 → 700	2	1	2	1	3	1
701 → 800	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0
901 → 1000	0	0	0	0	0	0
1001 → 1100	0	0	0	0	0	0
1101 → 1200	3	1	3	1	6	2
1201 → 1300	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0
1401 → 1500	0	0	2	1	3	1
1501 → 1600	0	0	0	0	0	0
1601 → 1700	0	0	1	1	2	1
1701 → 1800	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0
Total	5	2	8	4	14	5
Average Annual Failures	0.003	0.001	0.004	0.002	0.007	0.003

Table 5.5: Myponga Failure Occurrences for Various Inflow Exceedance Levels - (1)

Water Year Range	Myponga Failure Occurrence					
	60% Exceedance Inflow Forecasts		50% Exceedance Inflow Forecasts		40% Exceedance Inflow Forecasts	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
$1 \rightarrow 100$	0	0	0	0	0	0
$101 \rightarrow 200$	2	1	2	1	2	1
$201 \rightarrow 300$	0	0	0	0	0	0
$301 \rightarrow 400$	0	0	1	1	2	1
$401 \rightarrow 500$	0	0	0	0	0	0
$501 \rightarrow 600$	0	0	2	1	2	1
$601 \rightarrow 700$	5	2	7	2	7	2
$701 \rightarrow 800$	2	1	2	1	2	1
$801 \rightarrow 900$	0	0	0	0	0	0
$901 \rightarrow 1000$	0	0	0	0	0	0
$1001 \rightarrow 1100$	0	0	1	1	1	1
$1101 \rightarrow 1200$	9	4	13	5	20	7
$1201 \rightarrow 1300$	0	0	0	0	0	0
$1301 \rightarrow 1400$	0	0	0	0	1	1
$1401 \rightarrow 1500$	5	2	6	2	6	2
$1501 \rightarrow 1600$	3	2	6	3	6	3
$1601 \rightarrow 1700$	2	1	3	1	4	1
$1701 \rightarrow 1800$	1	1	1	1	1	1
$1801 \rightarrow 1900$	0	0	1	1	1	1
$1901 \rightarrow 2000$	0	0	0	0	0	0
Total	29	14	45	20	55	23
Average Annual Failures	0.015	0.007	0.023	0.010	0.028	0.012

Table 5.6: Myponga Failure Occurrences for Various Inflow Exceedance Levels - (2)

Water Year Range	Myponga Failure Occurrence					
	30% Exceedance Inflow Forecasts		20% Exceedance Inflow Forecasts		10% Exceedance Inflow Forecasts	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0
101 → 200	0	0	2	1	3	1
201 → 300	1	1	2	1	9	3
301 → 400	2	1	2	1	6	2
401 → 500	0	0	2	1	3	1
501 → 600	1	1	2	1	3	1
601 → 700	6	2	7	2	11	2
701 → 800	0	0	2	1	4	2
801 → 900	0	0	0	0	0	0
901 → 1000	0	0	0	0	1	1
1001 → 1100	1	1	2	1	4	2
1101 → 1200	12	5	21	8	37	10
1201 → 1300	1	1	1	1	2	1
1301 → 1400	0	0	0	0	1	1
1401 → 1500	5	2	6	2	9	2
1501 → 1600	5	3	10	4	15	5
1601 → 1700	4	1	4	1	10	3
1701 → 1800	1	1	7	3	17	5
1801 → 1900	1	1	1	1	6	3
1901 → 2000	0	0	0	0	1	1
Total	40	19	71	29	142	46
Average Annual Failures	0.020	0.010	0.036	0.015	0.071	0.023

Table 5.7: Myponga Failure Occurrences for Various Inflow Exceedance Levels - (3)

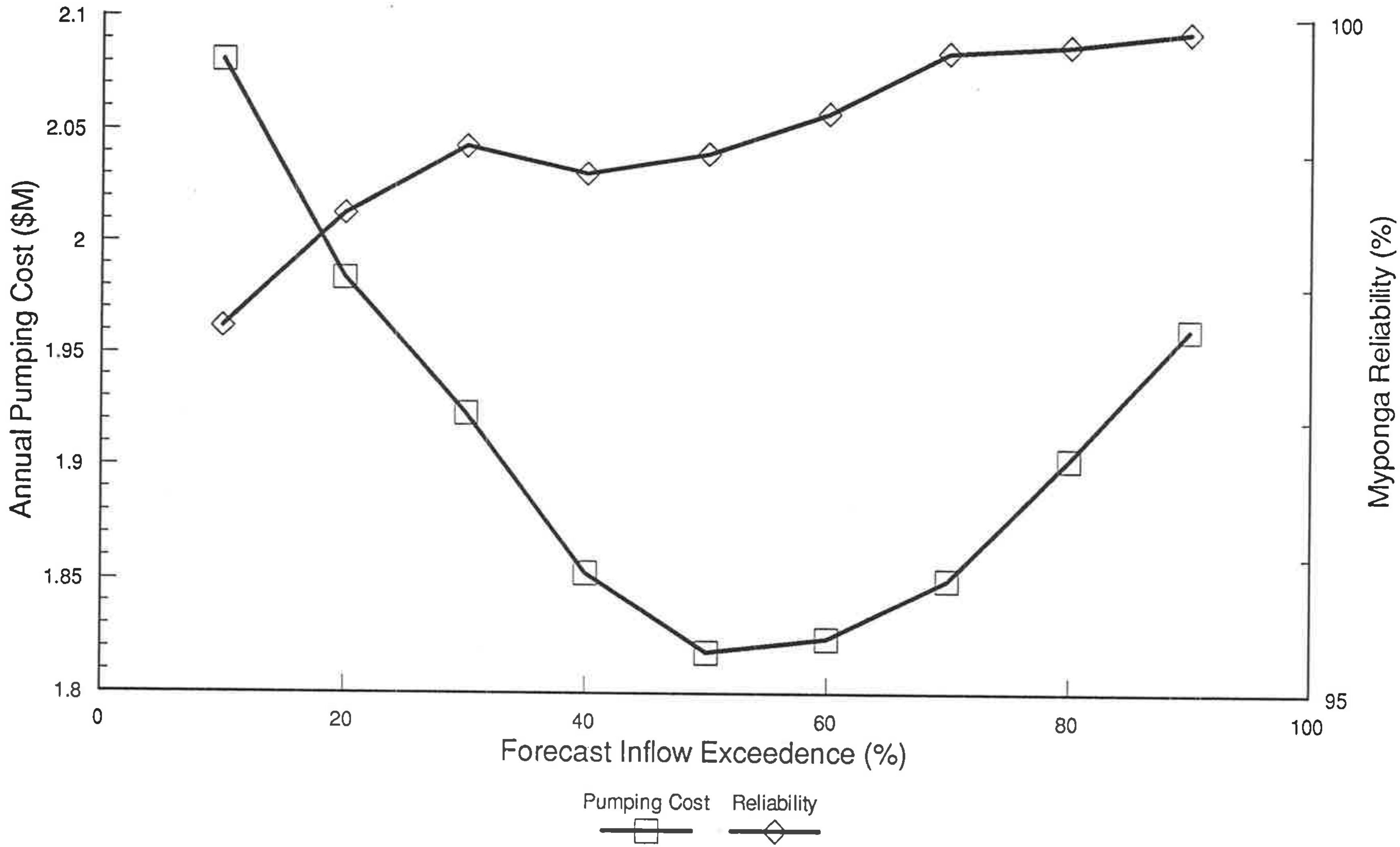


Figure 5.1: Southern System Inflow Exceedance Forecast - Reliability vs. Cost

Consideration of the results presented in Tables 5.2, 5.3 and 5.4, reveals that the minimum average operating cost for the system would be achieved using a 50% inflow exceedance forecast. This result is not unexpected as the 50% inflow exceedance values represent the median inflows for the system and therefore will provide the minimum cumulative difference between forecast and actual inflows.

Consideration of the failure occurrences for the Myponga Reservoir for each of the inflow exceedance forecasts reveals that as the inflow exceedance decreases the failure occurrences increase. This result is also intuitive and suggests that the inflow forecast sets should be selected in the range 90% to 50% according to the reliability requirements for the system.

Examination of the output obtained using the range of inflow exceedance sets reveals that total storage levels in the system are generally lower for lower inflow exceedance forecast sets. The distribution of these storages is not always consistent between Myponga and the Onkaparinga Reservoirs for different inflow exceedance forecast sets. Variations between the monthly pumping cost curves due to the Spring, Summer and Autumn electricity tariffs further complicate the situation. Differences in storages in the Onkaparinga system between runs using different inflow exceedance forecast sets are observed to result in minimum pumping costs being achieved with different distributions of monthly pumping with consequent differences in system reliability. The seemingly anomalous increase in reliability using 30% inflow exceedance sets rather than 40% inflow exceedance sets is an outcome of the combination of these factors. It is also noted that the annual pumping costs associated with the use of the 30% inflow exceedance sets falls slightly above a straight line falling between the 40% and 20% annual pumping cost figures.

Examination of Figure 5.1 showing Myponga Reservoir failure occurrence and pumping cost plotted against inflow exceedance forecasts suggests the selection of the 70% or 60% inflow exceedance as appropriate for the operation of the southern system. Although the minimum operating costs obtained for the operation of the southern system are obtained using 50% inflow exceedance sets examination of Figure 5.1 reveals that the reliability of the system drops significantly for inflow exceedance sets less than 70% with only minor reductions in system operating costs.

As a result of this examination, the 70% inflow exceedance sets have been adopted in this study for the operation of southern component of the Adelaide headworks system. This inflow exceedance data set has been used in all other considerations of the southern system described in later sections of this chapter.

5.2.1.2 Southern System - Demand Storage Level Comparison

As described in Chapter 4 in section 4.2.4.1, there are two components to the reservoir target storage levels adopted in the operating rules for the metropolitan Adelaide water supply system. These two components are the :

- Nominal minimum operating level component
- 'Demand storage' component

In this section, a comparison is made using a range of demand storage levels for the southern system using 8, 6, 4 and 2 weeks of demand as the 'demand storage' component of the reservoir target storage levels. Additional operating rules used in this comparison include the demand forecast set presented in Table 4.11 and the 70% exceedance forecast inflow sets presented in Tables 4.3 and 4.4. These operating rules have been examined using 2000 years of synthetic inflow and demand data. Details of the demand storage components of the target storage levels for the southern Adelaide system are presented in Appendix B in Tables B.5, B.6, B.7 and B.8. The nominal minimum operating levels components of the target storage levels used in this analysis are presented in Table B.4. Other details used in the HOMA simulation model are also given in Appendix B.

The streamflow and demand data has been generated using the data generation models previously described in Sections 4.4 and 4.5 of Chapter 4.

The comparative pumping costs obtained from each of the operating rule sets are given in Table 5.8. The months of failure and the number of failure events for the Myponga Reservoir are given in Table 5.9. These results have also been presented in graphical form in Figure 5.2. In all simulations of the system no failures occurred in either Mount Bold or Happy Valley Reservoirs.

The results highlight that supply failures from the Myponga Reservoir are critical to the operation of the southern system. The Onkaparinga system comprising the Mount Bold and Happy Valley Reservoirs is less critical in the operation of the system because of the capacity of the Murray Bridge-Onkaparinga pumping system relative to the southern system demand. If the capacity in this pumping system were reduced for an extended period of time, the target storage levels in Mount Bold and Happy Valley Reservoirs would influence the length of time before the southern system were to fail. The impact of the reliability of this pumping system is considered later in this chapter.

The results show that using '8 weeks demand' as the demand storage component for the reservoirs in the southern system results in a supply failure from Myponga Reservoir with a frequency of 1 in 400 years (99.75% reliability). These failures have an average duration of approximately 3 months.

For the synthetic data set considered, the demand storage component of the target storage levels could be lowered from '8 weeks demand' to '4 weeks demand' with a resulting reduction in average annual operating cost of approximately $135,000 per annum with no reduction in system reliability.

Water Year Range	Pumping Electricity Cost ($M)			
	8 Weeks Demand Target Storages	6 Weeks Demand Target Storages	4 Weeks Demand Target Storages	2 Weeks Demand Target Storages
1 → 100	166.456	156.532	151.644	145.042
101 → 200	174.385	165.255	159.713	152.775
201 → 300	201.875	193.497	188.835	182.456
301 → 400	176.086	167.829	163.395	157.280
401 → 500	192.148	183.738	179.277	172.766
501 → 600	166.308	156.788	151.924	146.144
601 → 700	196.352	187.102	181.946	175.527
701 → 800	192.352	183.556	179.278	173.177
801 → 900	173.900	164.675	159.041	152.069
901 → 1000	180.727	171.077	166.583	157.283
1001 → 1100	209.796	201.559	197.037	190.368
1101 → 1200	202.540	193.885	189.125	182.761
1201 → 1300	175.780	167.578	162.653	156.940
1301 → 1400	186.090	177.331	173.046	167.040
1401 → 1500	176.934	167.539	162.710	156.552
1501 → 1600	190.697	181.713	177.219	170.929
1601 → 1700	181.725	172.556	168.143	162.202
1701 → 1800	191.297	181.417	177.935	172.082
1801 → 1900	197.244	189.186	184.773	179.151
1901 → 2000	166.783	157.633	153.003	146.637
Total	3699.476	3526.446	3427.284	3299.181
Average Annual Cost	1.850	1.763	1.714	1.650

Table 5.8: Southern System Pumping Costs for Various Demand Storage Levels

Water Year Range	Myponga Failure Occurrence							
	8 Weeks Demand Target Storage		6 Weeks Demand Target Storage		4 Weeks Demand Target Storage		2 Weeks Demand Target Storage	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0	0	0
101 → 200	0	0	0	0	0	0	1	1
201 → 300	0	0	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0	0	0
601 → 700	3	1	3	1	3	1	3	1
701 → 800	0	0	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0	0	0
901 → 1000	0	0	0	0	0	0	0	0
1001 → 1100	0	0	0	0	0	0	0	0
1101 → 1200	6	2	6	2	6	2	6	2
1201 → 1300	0	0	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0	0	0
1401 → 1500	3	1	3	1	3	1	4	2
1501 → 1600	0	0	0	0	0	0	1	1
1601 → 1700	2	1	2	1	2	1	2	1
1701 → 1800	0	0	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0	0	0
Total	14	5	14	5	14	5	17	8
Average Annual Failures	0.007	0.003	0.007	0.003	0.007	0.003	0.009	0.004

Table 5.9: Myponga Failure Occurrences for Various Demand Storage Levels

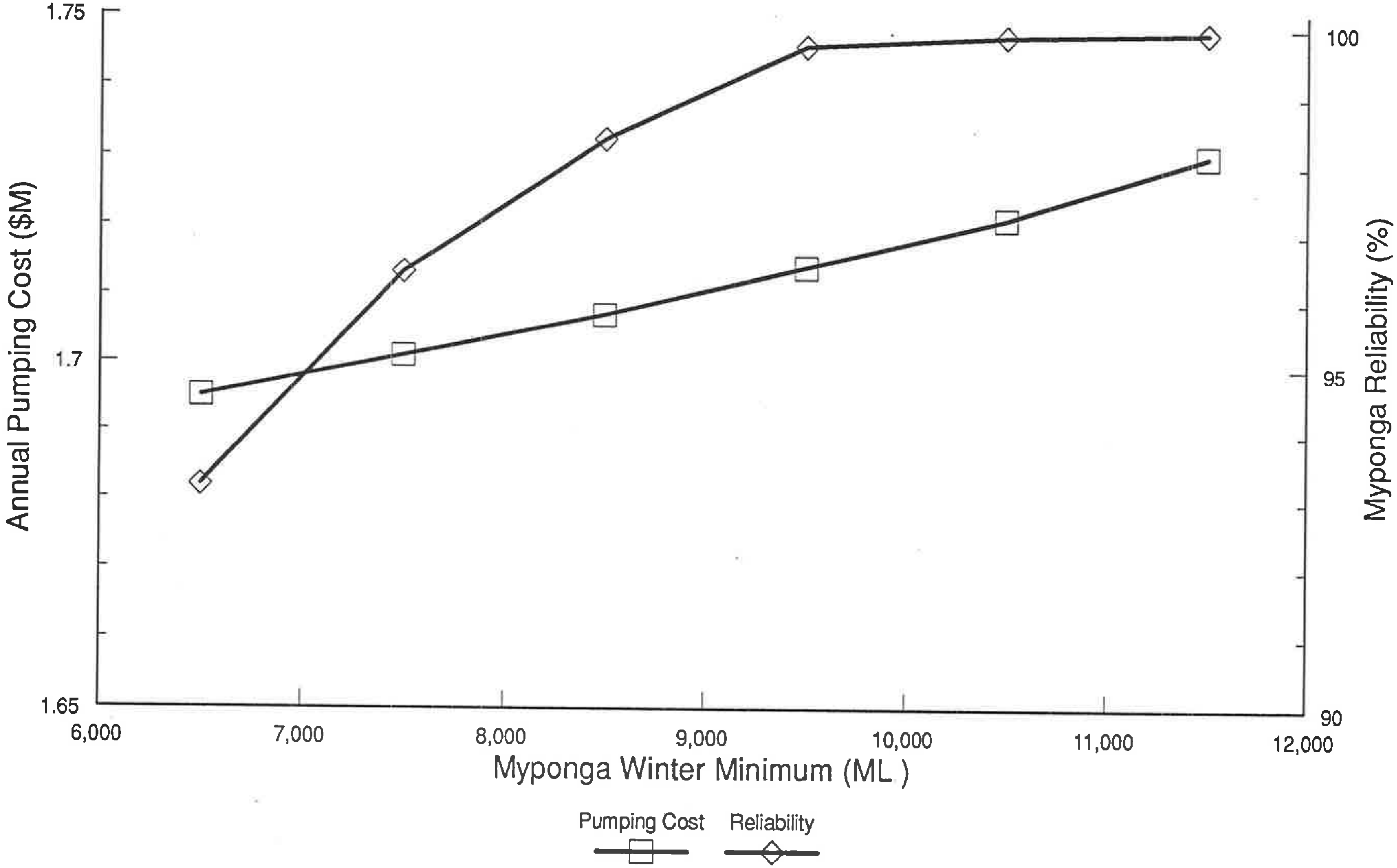


Figure 5.2: Southern System Demand Storage Level - Reliability vs. Cost

5.2.1.3 Southern System - Myponga Nominal Winter Minimum Operating Level Comparison

As highlighted in the previous two sections, the behaviour of the Myponga Reservoir has been identified as critical to the reliability of the southern system. Because this reservoir is unable to be supplemented with water from the River Murray, the target storage levels include a nominal winter minimum operating level for the months of May, June and July. This nominal winter minimum operating level has historically been set at 9,500 ML by EWS for Myponga Reservoir to accommodate 'two consecutive years of local demand, assuming 90% exceedance intakes'.

In this comparison the Myponga Reservoir nominal winter minimum operating level has been varied and the reliability-cost tradeoffs for the southern system examined using 2000 years of synthetic inflow and demand data. The 70% inflow exceedance sets given in Tables 4.3 and 4.4 and the '4 week demand' component of the target storages given in Table B.7 have been used in all simulations of the system. The demand forecast set adopted in this comparison is given in Table 4.11. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA simulation model are given in Appendix B.

The comparative pumping costs obtained for each of the operating rule sets considered are given in Tables 5.10 and 5.11. The corresponding failure occurrences in the Myponga Reservoir are also given in Tables 5.12 and 5.13. In all simulations of the system no failures occurred in Mount Bold or Happy Valley Reservoirs.

Water Year Range	Pumping Electricity Cost ($M)		
	11500 ML Winter Min.	10500 ML Winter Min.	9500 ML Winter Min.
1 → 100	152.982	152.188	151.644
101 → 200	161.201	160.459	159.713
201 → 300	190.768	189.754	188.835
301 → 400	165.160	164.294	163.395
401 → 500	180.885	180.061	179.277
501 → 600	153.355	152.544	151.924
601 → 700	183.930	182.897	181.946
701 → 800	180.731	179.982	179.278
801 → 900	160.712	159.891	159.041
901 → 1000	167.892	167.215	166.583
1001 → 1100	199.165	198.015	197.037
1101 → 1200	190.841	189.972	189.125
1201 → 1300	164.270	163.394	162.653
1301 → 1400	174.268	173.652	173.046
1401 → 1500	164.356	163.485	162.710
1501 → 1600	178.831	178.014	177.219
1601 → 1700	169.736	168.925	168.143
1701 → 1800	179.860	178.841	177.935
1801 → 1900	186.254	185.436	184.773
1901 → 2000	154.479	153.728	153.003
Total	3459.676	3442.747	3427.284
Average Annual Cost	1.730	1.721	1.714

Table 5.10: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels - (1)

Water Year Range	Pumping Electricity Cost ($M)		
	8500 ML Winter Min.	7500 ML Winter Min.	6500 ML Winter Min.
1 → 100	150.989	150.580	150.204
101 → 200	159.063	158.552	158.057
201 → 300	187.986	187.234	186.418
301 → 400	162.600	162.021	161.374
401 → 500	178.635	178.161	177.557
501 → 600	151.422	151.035	150.542
601 → 700	181.065	180.377	179.429
701 → 800	178.618	178.053	177.335
801 → 900	158.326	157.716	156.851
901 → 1000	166.046	165.493	164.786
1001 → 1100	196.204	195.512	194.811
1101 → 1200	188.318	187.635	186.869
1201 → 1300	162.143	161.688	161.023
1301 → 1400	172.583	172.156	171.694
1401 → 1500	162.028	161.410	160.768
1501 → 1600	176.388	175.683	174.934
1601 → 1700	167.417	166.757	166.092
1701 → 1800	177.227	176.610	175.853
1801 → 1900	184.041	183.280	182.376
1901 → 8000	152.363	151.778	151.099
Total	3413.462	3401.731	3388.072
Average Annual Cost	1.707	1.701	1.695

Table 5.11: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels - (2)

Water Year Range	Myponga Failure Occurrence					
	11500 ML Winter Min.		10500 ML Winter Min.		9500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0
101 → 200	0	0	0	0	0	0
201 → 300	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0
601 → 700	1	1	2	1	3	1
701 → 800	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0
901 → 1000	0	0	0	0	0	0
1001 → 1100	0	0	0	0	0	0
1101 → 1200	0	0	3	1	6	2
1201 → 1300	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0
1401 → 1500	0	0	0	0	3	1
1501 → 1600	0	0	0	0	0	0
1601 → 1700	0	0	0	0	2	1
1701 → 1800	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0
Total	1	1	5	2	14	5
Average Annual Failures	0.0005	0.0005	0.0025	0.001	0.007	0.0025
Lowest Myponga Reservoir Level attained (ML)	4365		3884		3387	

Table 5.12: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels - (1)

Water Year Range	Myponga Failure Occurrence					
	8500 ML Winter Min.		7500 ML Winter Min.		6500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	8	5
101 → 200	2	1	7	3	12	4
201 → 300	1	1	8	3	25	9
301 → 400	2	2	9	4	13	4
401 → 500	0	0	6	4	18	7
501 → 600	2	1	3	1	8	2
601 → 700	7	2	10	3	33	15
701 → 800	3	1	10	4	36	11
801 → 900	0	0	0	0	2	1
901 → 1000	0	0	2	1	10	3
1001 → 1100	7	4	10	4	25	8
1101 → 1200	24	7	47	13	85	19
1201 → 1300	0	0	2	1	10	4
1301 → 1400	3	1	7	3	11	3
1401 → 1500	7	2	13	3	20	4
1501 → 1600	6	3	19	6	31	6
1601 → 1700	5	3	17	6	28	6
1701 → 1800	3	2	18	6	48	12
1801 → 1900	3	2	12	5	33	9
1901 → 2000	0	0	2	1	8	3
Total	75	32	202	71	464	135
Average Annual Failures	0.0375	0.016	0.101	0.0355	0.232	0.0675
Lowest Myponga Reservoir Level attained (ML)	2509		1871		970	

Table 5.13: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels - (2)

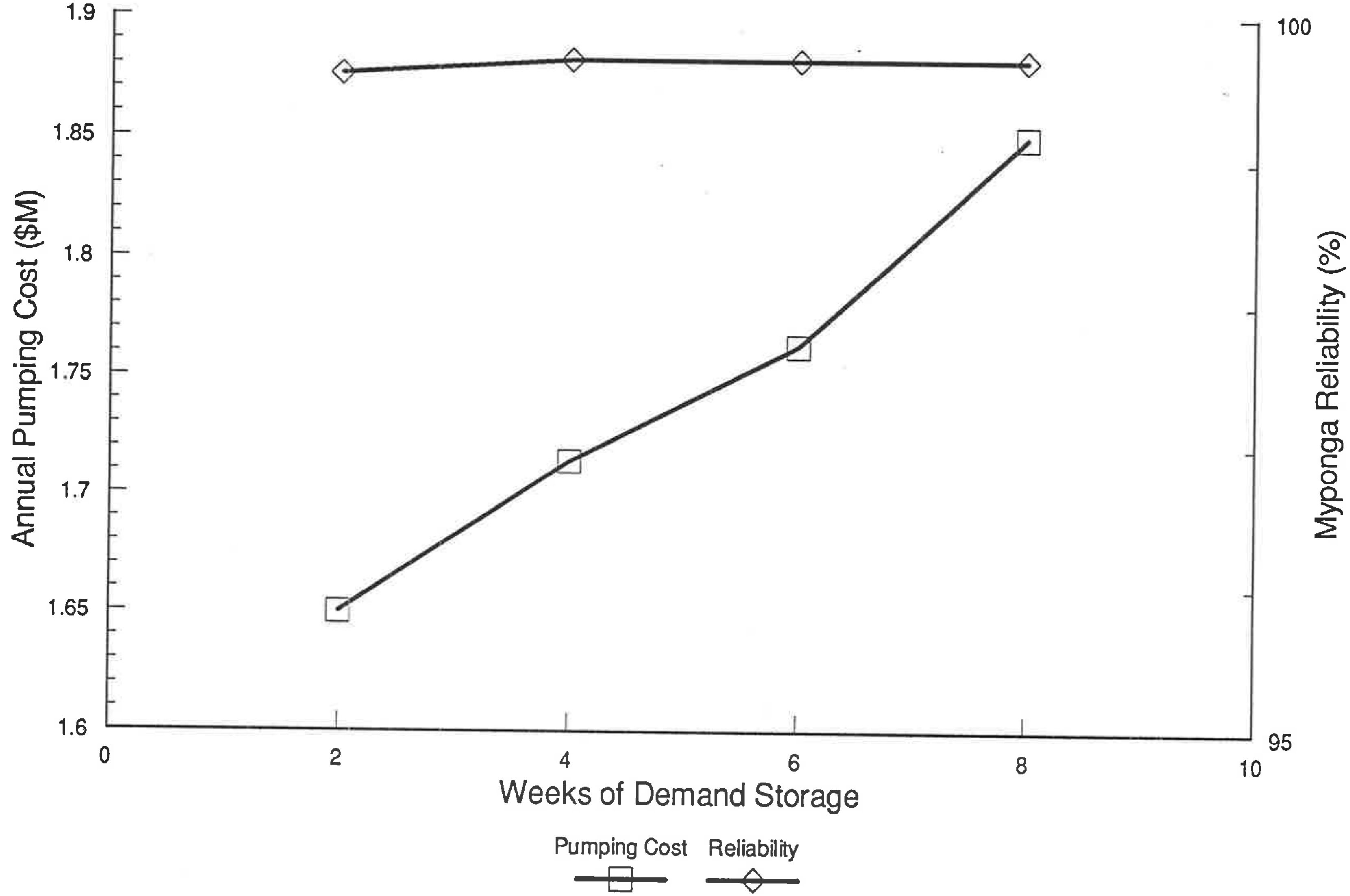


Figure 5.3: Southern System Myponga Nominal Winter Minimum Operating Level - Reliability vs. Cost

Consideration of the results presented in Tables 5.10 to 5.11 highlights the tradeoff between operating cost and failure occurrence for Myponga Reservoir when the Myponga Reservoir nominal winter minimum operating level is varied. These results have been plotted in Figure 5.3. The occurrence of failures in Myponga Reservoir increases dramatically with a reduction in the reservoir nominal winter minimum operating level. Using the tradeoffs between reliability and operating costs presented in Figure 5.3, an acceptable Myponga nominal winter minimum operating level can be selected by the system operators.

5.2.1.4 Southern System - Onkaparinga System Nominal Winter Minimum Operating Level Comparison

In this section, the effect of varying the Mount Bold and Happy Valley Reservoir nominal winter minimum operating levels has been considered on the reliability-cost tradeoffs for the southern system using 2000 years of synthetic inflow and demand data.

In this comparison, a Myponga nominal winter minimum level of 9500 ML has been adopted, in conjunction with the 70% exceedance inflow forecast sets. The demand component of the target storage levels in all reservoirs in the system has been set at '4 weeks demand' and the demand forecast set given in Table 4.11 has been used. Other details used in the HOMA simulation model are given in Appendix B.

As previously described, the Onkaparinga system comprises the Mount Bold and Happy Valley Reservoirs. These two reservoirs are operated in conjunction. As additional water is required at Happy Valley Reservoir, water is released from Mount Bold Reservoir to Clarendon Weir and transferred into Happy Valley Reservoir as described in Section 4.2 of Chapter 4. In practice, this operation is carried out on a daily basis. Within this examination it is intended to show that lowering the nominal winter operating level sufficiently can result in a failure occurring in the Onkaparinga system. To achieve this purpose a HOMA simulation is presented with an Onkaparinga system nominal winter minimum operating level given as -100 ML,

Since the operation of the Mount Bold and Happy Valley Reservoirs are so closely linked, this run has been achieved by setting the Happy Valley Reservoir nominal winter minimum operating level to zero and reducing the Mount Bold Reservoir nominal winter minimum operating level to -100 ML. The additional 100 ML is obtained from the '4 week demand' component of the target storage level.

The comparative pumping costs are given in Tables 5.14 and 5.15. The corresponding failure occurrences in the Onkaparinga system are shown in Table 5.16. Only when the Onkaparinga System nominal winter minimum operating levels was reduced to -100 ML did failures occur in the Onkaparinga system.

The results given in Tables 5.14 and 5.15 are presented graphically in Figure 5.4.

Water Year Range	Pumping Electricity Cost ($M) 4900 ML Winter Min.	3900 ML Winter Min.	2900 ML Winter Min.
1 → 100	150.989	149.451	146.953
101 → 200	159.063	157.772	155.247
201 → 300	187.986	187.443	185.146
301 → 400	162.600	161.867	159.495
401 → 500	178.635	177.584	175.048
501 → 600	151.422	150.211	147.951
601 → 700	181.065	180.278	178.029
701 → 800	178.618	177.666	175.395
801 → 900	158.326	157.182	154.547
901 → 1000	166.046	164.498	162.265
1001 → 1100	196.204	195.439	192.840
1101 → 1200	188.318	187.604	185.415
1201 → 1300	162.143	161.177	159.060
1301 → 1400	172.583	171.305	169.130
1401 → 1500	162.028	161.061	158.680
1501 → 1600	176.388	175.684	173.451
1601 → 1700	167.417	166.562	164.259
1701 → 1800	177.227	176.468	174.178
1801 → 1900	184.041	183.250	181.145
1901 → 2000	152.363	151.068	148.831
Total	3413.462	3393.580	3347.065
Average Annual Cost	1.707	1.697	1.674

Table 5.14: Southern System Pumping Costs for Various Onkaparinga Nominal Winter Minimum Operating Levels - (1)

Water Year Range	Pumping Electricity Cost ($M)		
	1900 ML Winter Min.	900 ML Winter Min.	-100 ML Winter Min.
1 → 100	144.540	142.218	140.172
101 → 200	152.768	150.362	148.003
201 → 300	182.806	180.561	178.409
301 → 400	157.299	155.337	153.580
401 → 500	172.632	170.282	168.065
501 → 600	145.791	143.752	141.696
601 → 700	175.913	173.785	171.762
701 → 800	173.223	171.127	168.924
801 → 900	152.054	149.668	147.473
901 → 1000	159.862	157.482	155.209
1001 → 1100	190.289	187.887	185.511
1101 → 1200	183.233	181.053	178.994
1201 → 1300	157.034	155.092	153.173
1301 → 1400	167.018	164.973	163.017
1401 → 1500	165.347	154.110	152.102
1501 → 1600	171.240	169.241	167.175
1601 → 1700	161.968	159.633	157.450
1701 → 1800	171.898	169.748	167.655
1801 → 1900	179.066	177.163	175.339
1901 → 2000	145.991	143.701	141.434
Total	3300.972	3257.175	3215.133
Average Annual Cost	1.650	1.629	1.608

Table 5.15: Southern System Pumping Costs for Various Onkaparinga Nominal Winter Minimum Operating Levels - (2)

Water Year Range	Onkaparinga Failure Occurrence					
	1900 ML Winter Min.		900 ML Winter Min.		-100 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	43	29
101 → 200	0	0	0	0	53	32
201 → 300	0	0	0	0	58	37
301 → 400	0	0	0	0	47	30
401 → 500	0	0	0	0	61	39
501 → 600	0	0	0	0	46	30
601 → 700	0	0	0	0	50	31
701 → 800	0	0	0	0	57	32
801 → 900	0	0	0	0	39	25
901 → 1000	0	0	0	0	56	35
1001 → 1100	0	0	0	0	56	34
1101 → 1200	0	0	0	0	58	38
1201 → 1300	0	0	0	0	46	30
1301 → 1400	0	0	0	0	63	40
1401 → 1500	0	0	0	0	62	37
1501 → 1600	0	0	0	0	59	36
1601 → 1700	0	0	0	0	38	25
1701 → 1800	0	0	0	0	50	34
1801 → 1900	0	0	0	0	65	39
1901 → 2000	0	0	0	0	48	30
Total	0	0	0	0	1055	663
Average Annual Failures	0.000	0.000	0.000	0.000	0.528	0.332

Table 5.16: Onkaparinga Failure Occurrences for Various Onkaparinga Nominal Winter Minimum Operating Levels

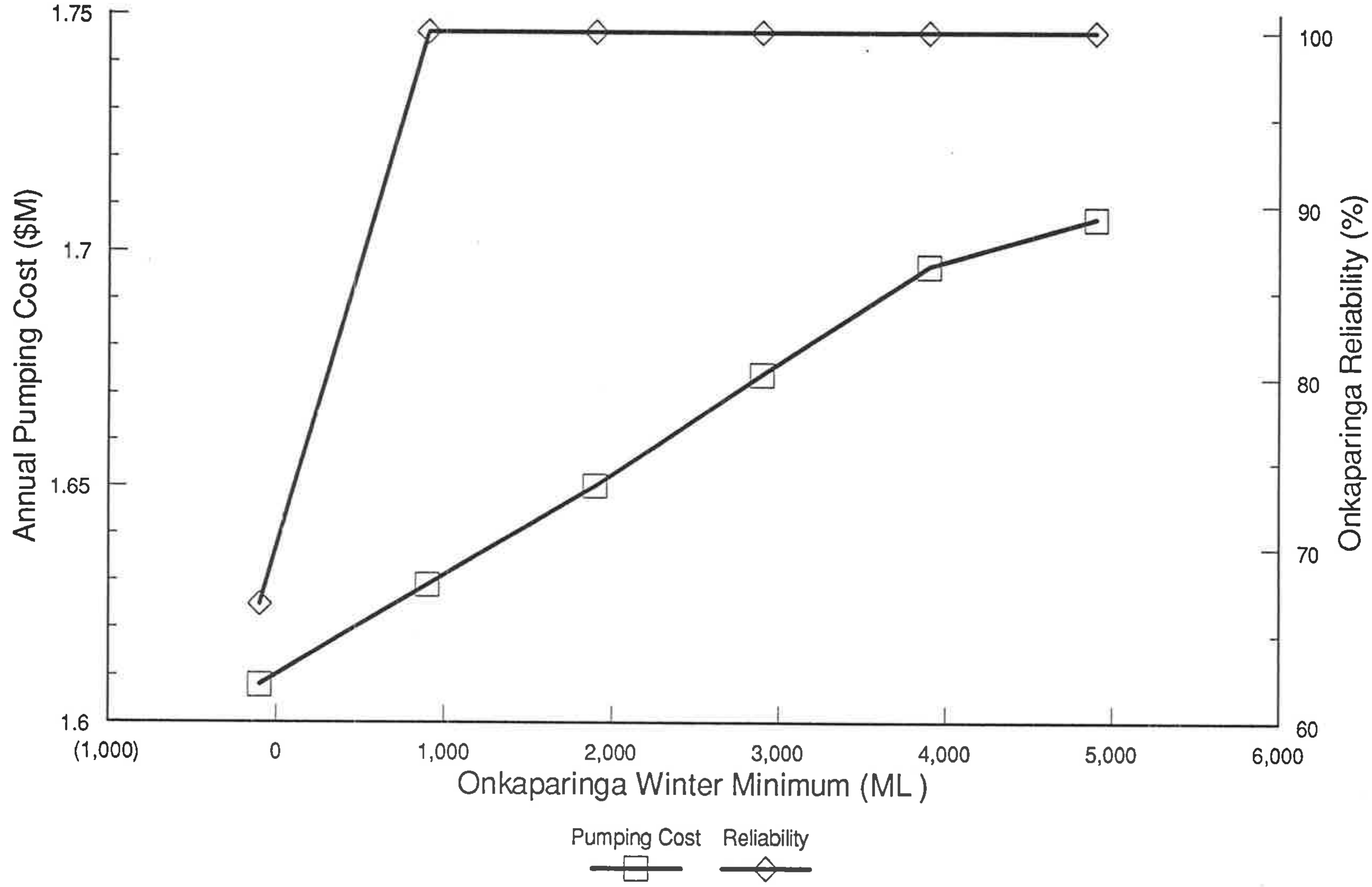


Figure 5.4: Southern System Onkaparinga Nominal Winter Minimum Operating Level vs. Cost

For each of the Onkaparinga nominal winter minimum operating levels considered, five months of failure occurred in the Myponga Reservoir comprising two events. Over the 2000 year simulation period considered this represents a reliability of 99.9%. No failures occurred in the Onkaparinga system until the nominal winter minimum operating level for the Onkaparinga system was set to -100 ML. These results show that the capacity of pumping on the Murray Bridge Onkaparinga pumping system is sufficient to meet demands from Happy Valley Reservoir throughout the year with only small balancing storages required in the Onkaparinga reservoirs. The nominal winter minimum operating level for the Onkaparinga system should therefore be selected with consideration of the reliability of the Murray Bridge-Onkaparinga pumping system.

In conclusion, consideration of the reliability-cost tradeoffs for southern Adelaide water supply system, taking into account hydrologic variability has revealed the potential for significant reductions in operating costs with little or no loss in system reliability. The dependence of the southern system on pumping from the River Murray via the Murray Bridge-Onkaparinga pumping system, necessitates the inclusion of the impact of the reliability of this pumping system in the examination of reliability-cost tradeoffs. The impact of the reliability of this system is examined later in section 5.3 of this chapter.

5.2.2 Northern System

The following sections detail reliability-cost tradeoffs for the northern system taking into account hydrologic variability associated with the operation of the system. These tradeoffs have been determined by considering the application of a range of system operating rules. In these comparisons, the physical minimum operating levels below which failure is assumed to occur in the northern system reservoirs is given in Table 5.17. A description of the physical minimum operating level for a reservoir has previously been given in Section 4.2.4.1.

Reservoir	Physical Minimum Operating Level (ML)
Warren	0
South Para	1100
Barossa	3100
Little Para	3100
Millbrook	300
Kangaroo Creek	100
Hope Valley	1200

Table 5.17: Northern System Physical Minimum Reservoir Operating Levels

In the case of South Para, Barossa, Little Para and Hope Valley Reservoirs, it is possible for these reservoirs to be drawn below their physical minimum operating levels in a crisis situation with the installation of temporary pumps and associated pipework.

In the simulation of the northern system it has been assumed that short-term draw-down of these reservoirs may occur in order to satisfy demand. When reservoir levels are drawn down below these levels, a 'failure' is assumed to have occurred, however full demand requirements are assumed to have been met.

5.2.2.1 Northern System - Inflow Exceedance Comparison

As described in section 4.2.4.2 of Chapter 4, a forecast set of inflow and demand volumes are assumed at the commencement of the water year in the preparation of the pumping program for the coming year. As the year progresses, these forecast inflows and demands are updated with 'actual' inflow and demand volumes and the pumping program for the remainder of the year is reformulated.

In this section a comparison is made for the northern system using forecast inflow data sets having 90% to 40% inflow exceedance values. Details of these inflow exceedance data sets for the northern system are given in Chapter 4 in Tables 4.5 to 4.10. 30% to 10% inflow exceedance data sets have not been considered, as results from the southern system highlight the use of these data sets will result in higher operating costs with reduced reliability. Additional operating rules used in this comparison include the demand forecast set presented in Table 4.12 and target storage levels for the reservoirs in the system, comprising the nominal minimum reservoir operating levels given in Table B.14 plus '8 weeks demand' given in Tables B.15 and B.16. Other details used in the HOMA simulation model are given in Appendix B.

A synthetic inflow and demand data record of 1000 years has been used to examine the long term operating and failure behaviour of the system using this range of operating rule sets. A 1000 year record set has been used for the northern system in contrast to the 2000 year record set for the southern system because of the greater complexity of the northern system and hence longer computer run times for the simulations. Simulation of 1000 years of record of the northern system using the HOMA model in 'forecast' mode on a Sun Sparcstation 20 takes approximately 12.5 hours.

This data has been generated using the streamflow and demand data generation models previously described in Chapter 4.

The comparative pumping costs are presented in Tables 5.18 and 5.19.

There were no failure occurrences in any reservoir in the northern system for any of the operating rule sets examined.

Consideration of the results shown in Tables 5.18 and 5.19, and plotted in Figure 5.5 reveals that the minimum average operating cost for the northern system occurs using a 40% inflow exceedance forecast. The results obtained

Water Year Range	Pumping Electricity Cost ($M)		
	90% Exceedance Inflow Forecasts	80% Exceedance Inflow Forecasts	70% Exceedance Inflow Forecasts
1 → 100	498.820	487.581	478.662
101 → 200	493.306	480.617	468.908
201 → 300	551.876	545.561	539.404
301 → 400	525.310	514.475	505.327
401 → 500	527.645	517.614	507.788
501 → 600	510.833	499.194	492.505
601 → 700	555.703	542.836	537.825
701 → 800	555.699	549.281	540.419
801 → 900	507.963	495.696	486.622
901 → 1000	516.901	505.878	497.872
Total	5244.056	5138.733	5055.332
Average Annual Cost	5.244	5.139	5.055

Table 5.18: Northern System Pumping Costs for Various Inflow Exceedance Levels - (1)

Water Year Range	Pumping Electricity Cost ($M)		
	60% Exceedance Inflow Forecasts	50% Exceedance Inflow Forecasts	40% Exceedance Inflow Forecasts
1 → 100	470.726	465.144	465.990
101 → 200	458.868	451.380	449.006
201 → 300	535.285	530.077	535.468
301 → 400	497.381	488.435	484.134
401 → 500	498.066	489.790	489.093
501 → 600	484.684	476.638	477.921
601 → 700	528.058	523.992	523.200
701 → 800	532.196	525.412	525.213
801 → 900	479.425	474.216	474.577
901 → 1000	487.301	483.085	480.379
Total	4971.990	4908.169	4904.513
Average Annual Cost	4.972	4.908	4.905

Table 5.19: Northern System Pumping Costs for Various Inflow Exceedance Levels - (2)

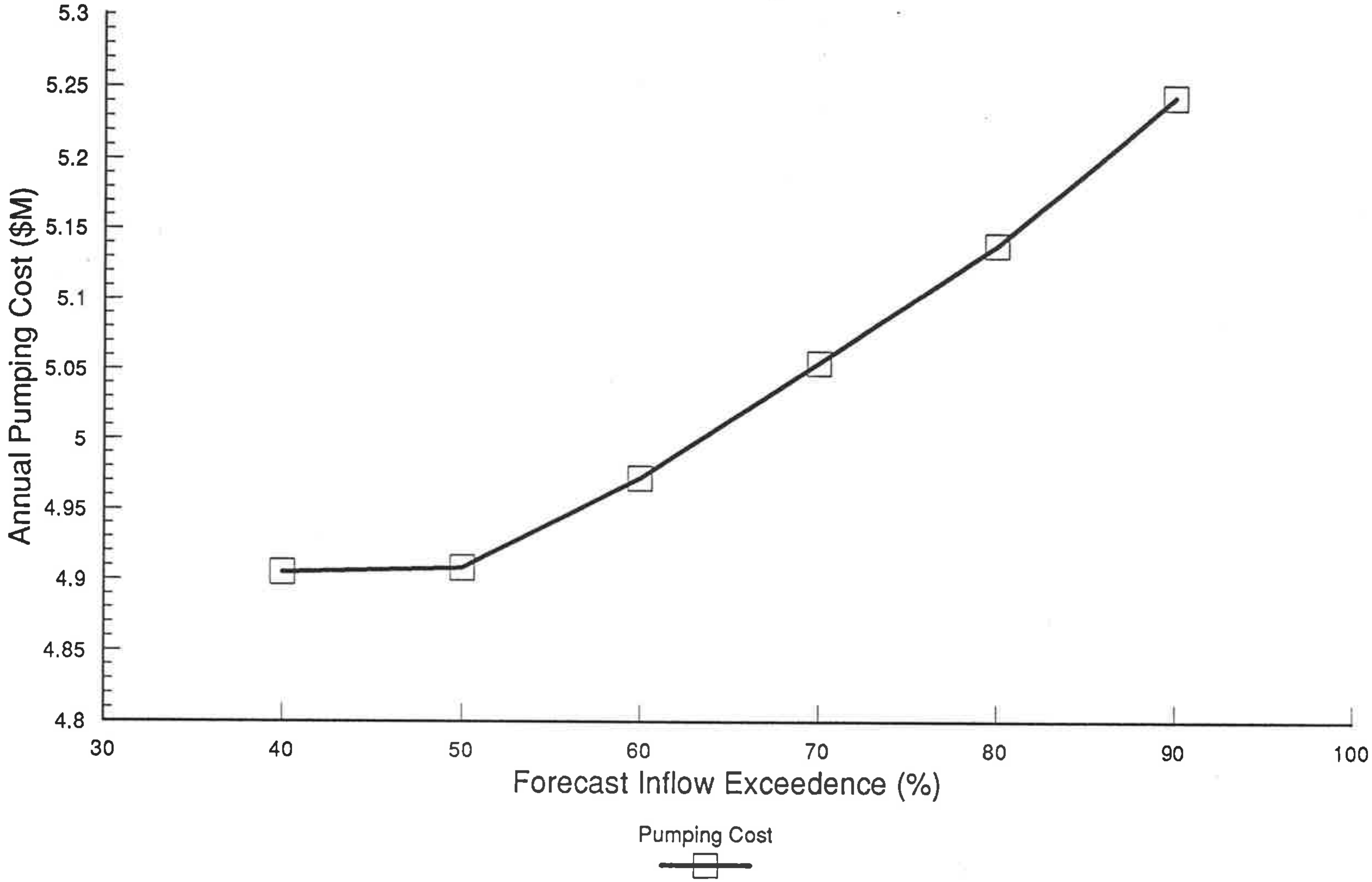


Figure 5.5: Northern System Inflow Forecast - Exceedance vs. Cost

using a 50% inflow exceedance forecast set are marginally higher than the 40% exceedance case. Although lower exceedance inflow data sets have not been considered, results from the southern system indicate that the use of these sets would result in higher operating costs, at a reduced reliability level.

It was initially expected that the minimum annual average operating costs would be obtained using the 50% inflow exceedance sets for the northern system. Detailed examination of the output files reveals a number of contributing factors resulting in the 40% inflow exceedance sets annual average operating costs being marginally lower. These factors include :

- Consideration of the historical mean and median statistics for the southern and northern system inflow gauging stations presented in Tables 4.14, 4.15, 4.16, 4.17 and 4.18 of Chapter 4 reveals that the ratio of winter mean monthly inflows to median monthly inflows is generally greater for the northern system gauging stations than the southern system gauging stations. The 50% inflow exceedance set represents the median rather than mean inflow set. The minimum operating costs are likely to occur when the inflow forecast set is closest to the mean inflow set. Since the mean inflow set is closer to the 40% exceedance set than the 50% inflow set for the northern system gauging stations, the results for the 40% exceedance set will be slightly lower than the results for the 50% inflow set.

- The use of the 40% inflow exceedance sets results in the increased usage of the Millbrook pump station. As the Torrens system approaches full capacity, Millbrook pump station can be used to transfer water to the Little Para Reservoir or directly supply metropolitan Adelaide through Anstey Hill Water Filtration Plant. Using the lower inflow exceedance set results in increased scheduling of this pump station as the forecast inflow to Millbrook Reservoir will be greater. In the event that the actual inflows do not attain the forecast inflows, Millbrook Reservoir will not reach the forecast storage level. In the event that the actual inflows exceed the forecast inflows and the Torrens system is at or near capacity, the volumes of spill will be lower if the 40% inflow exceedance sets has been assumed.

- The use of the 40% inflow exceedance sets results in all reservoirs in the northern system reaching lower minimum levels during the period of simulation. Since these minimum reservoir levels are still above the failure levels for the system, no failures are recorded. The reliability level using the 40% inflow exceedance sets will be marginally lower than that obtained using the 50% inflow exceedance sets. The reduction in reservoir storage levels will also have benefits in terms of reduced evaporation losses.

- In those situations when the inflow is less than the forecast inflow set, and additional pumping is required, the additional pumping will be undertaken at the next months marginal pumping cost. When the 40% inflow exceedance sets is applied, a slight increase in pumping costs will result above that obtained with the use of the 50% inflow exceedance sets. This effect will counter-balance pumping cost reductions obtained through the previous two factors.

The summation of the factors affecting the pumping costs for the northern system using the 40% inflow exceedance sets in contrast to the 50% inflow exceedance sets result in an average annual operating cost of \$4.905M compared to \$4.908M.

Results from the inflow exceedance analysis for the southern system, presented earlier in this section, resulted in the adoption of the 70% inflow exceedance sets. For consistency with the analysis of the southern system, the 70% inflow exceedance sets have also been adopted and used in all other considerations of the northern system described in later sections. It is noted that further potential reduction in operating costs with no apparent loss in system reliability could be obtained through the application of the 50% inflow exceedance sets in place of the 70% inflow exceedance sets.

5.2.2.2 Northern System - Demand Storage Level Comparison

As described in Chapter 4 in section 4.2.4.1 there are two components to the reservoir target storage levels adopted in the operating rules for the metropolitan Adelaide water supply system. These two components are the :

- Nominal minimum operating level component
- 'Demand storage' component

In this section a comparison is made between a range of operating rules for the northern system using runs with 8, 6, 4 and 2 weeks of demand as the 'demand storage' component of the reservoir target storage levels.

For each of the four cases considered, the 70% inflow exceedance data sets for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. These operating rules are examined using 1000 years of synthetic inflow and demand data. Details of the components comprising the target storage levels for the northern Adelaide system are presented in Appendix B in Tables B.15, B.17, B.19 and B.22. The nominal minimum operating levels components of the target storage levels used in this analysis are presented in Table B.14. Other details used in the HOMA simulation model are given in Appendix B.

The streamflow and demand data has been generated using the generation models previously described in Sections 4.4 and 4.5 of Chapter 4.

The comparative pumping costs obtained from this comparison are presented in Table 5.20.

Water Year Range	Pumping Electricity Cost ($M)			
	8 Weeks Demand Target Storages	6 Weeks Demand Target Storages	4 Weeks Demand Target Storages	2 Weeks Demand Target Storages
1 → 100	478.662	463.655	455.799	444.810
101 → 200	468.908	454.814	446.163	434.614
201 → 300	539.404	524.633	517.482	509.043
301 → 400	505.327	492.483	482.386	473.144
401 → 500	507.788	491.117	482.606	471.722
501 → 600	492.505	477.159	469.082	458.753
601 → 700	537.825	522.763	514.964	504.905
701 → 800	540.419	523.436	516.579	506.876
801 → 900	486.622	472.353	463.493	453.980
901 → 1000	497.872	481.966	472.244	463.346
Total	5055.332	4904.379	4820.798	4721.193
Average Annual Cost	5.055	4.904	4.821	4.721

Table 5.20: Northern System Pumping Costs for Various Demand Storage Levels

No system failures occurred during any of the 1000 year simulations of the northern system for the operating rule sets considered.

The results indicate that the current pumping capacity and minimum storage levels in the northern system are adequate to accommodate any hydrological shortfalls even when the demand storage component of the target storage values are reduced to 'two weeks demand' above the nominal minimum storage levels.

For the simulation period considered, the demand storage component of the target storage levels could be lowered from '8 weeks demand' to '4 weeks demand' with a resulting reduction in average annual operating cost of approximately $234,000 with no reduction in system reliability. Further reduction of the demand storage component to '2 weeks demand' would result in an additional annual reduction in operating costs of $100,000.

Results from the demand storage analysis for the southern system presented earlier in this section resulted in the adoption of a '4 weeks demand' component of the target storage levels for the southern system by the EWS. It was decided in this study, for consistency, that this demand storage component of the target storage levels should also be adopted for the northern system.

The '4 weeks demand' component of the target storage levels has been used in all other considerations of the northern system described in later sections.

The operation of the northern system is highly dependent on pumping from the River Murray via the Mannum-Adelaide and Swan Reach Stockwell pipelines and pumping at the Millbrook pump station. The overall reliability of the system is therefore dependent on the reliability of the critical components of these pumping systems. The impact of the reliability of these components is considered later in Section 5.3 of this chapter.

5.2.2.3 Northern System - South Para Nominal Winter Minimum Operating Level Comparison

In this section, a comparison is made for the operating rules for the South Para subsystem of the northern system. The nominal winter minimum operating level currently adopted in the operation of the northern system is 11,100 ML. Following discussion with a number of personnel within the EWS, the basis for the selection of this nominal winter minimum operating level is unclear, but is thought to relate to the reliability of the system.

In this comparison, the nominal winter minimum operating level component of the target storage levels for the South Para Reservoir have been varied and a synthetic inflow and demand data record of 1000 years has been used to examine the long term operating behaviour for the system under these operating rules. For each of the nominal winter minimum operating level components of the target storage levels, 4 weeks of demand storage has been used to obtain the monthly target storage levels. For each of the six cases considered, the 70% inflow exceedance data set for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. For all other reservoirs, the nominal minimum operating level components of the target storage levels used in this analysis are those presented in Table B.14. Other details used in the HOMA model are given in Appendix B.

The comparative pumping costs are given in Tables 5.21 and 5.22.

Water Year Range	Pumping Electricity Cost ($M)		
	11100 ML Winter Min.	10100 ML Winter Min.	8100 ML Winter Min.
1 → 100	455.799	454.327	451.008
101 → 200	446.163	444.035	440.316
201 → 300	517.482	515.988	512.083
301 → 400	482.386	481.488	478.221
401 → 500	482.606	480.863	476.760
501 → 600	469.082	466.970	463.354
601 → 700	514.964	512.557	509.446
701 → 800	516.579	515.012	511.797
801 → 900	463.493	461.753	458.234
901 → 1000	472.244	470.523	468.064
Total	4820.798	4803.516	4769.283
Average Annual Cost	4.821	4.804	4.769

Table 5.21: Northern System Pumping Costs for Various South Para Nominal Winter Minimum Operating Levels - (1)

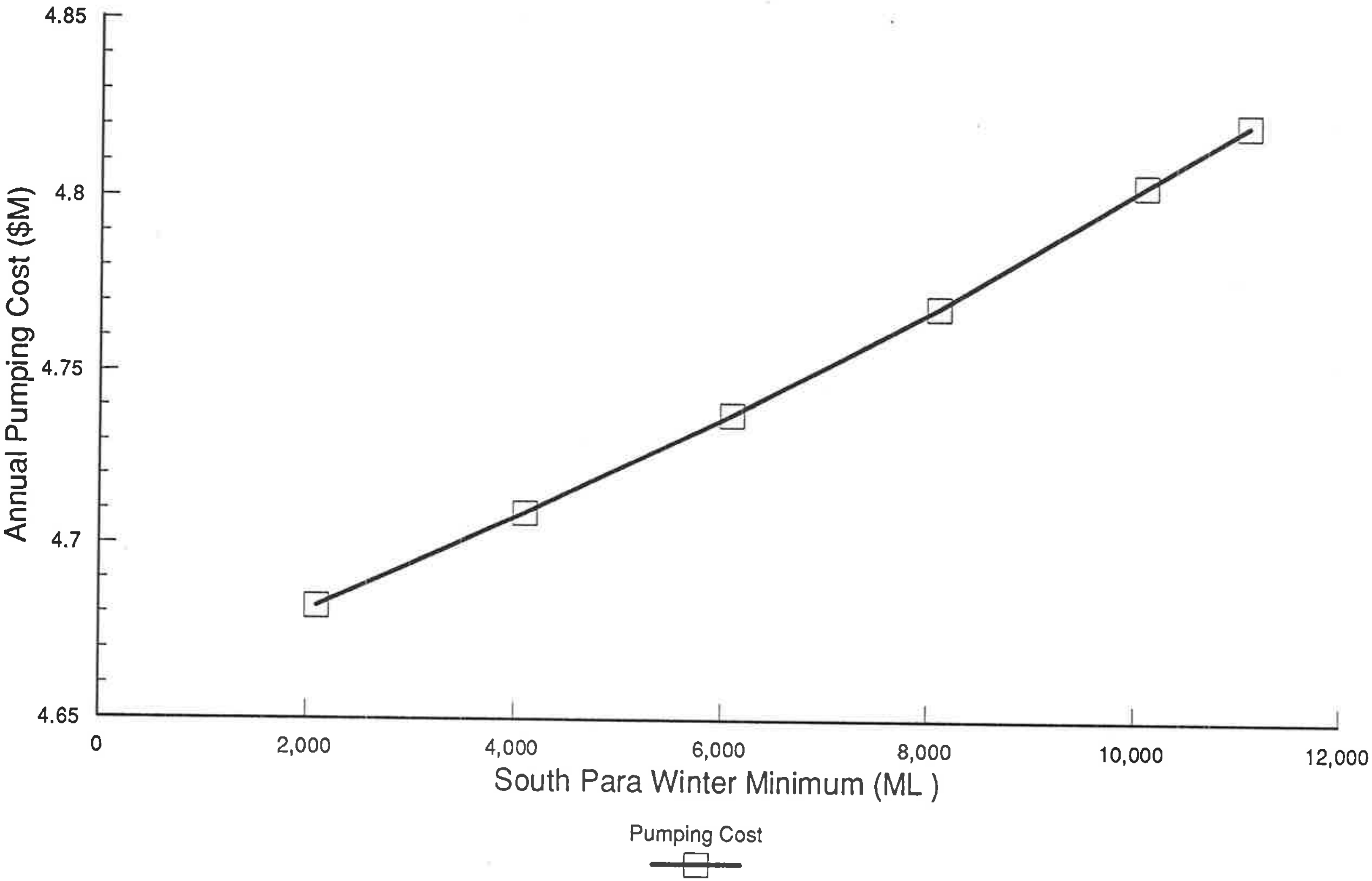


Figure 5.6: Northern System South Para Nominal Winter Minimum Operating Level vs. Cost

Water Year Range	Pumping Electricity Cost ($M)		
	6100 ML Winter Min.	4100 ML Winter Min.	2100 ML Winter Min.
1 → 100	447.801	444.984	442.150
101 → 200	437.124	434.286	431.705
201 → 300	508.955	506.109	503.244
301 → 400	475.247	471.504	468.925
401 → 500	473.235	469.730	466.484
501 → 600	460.065	457.162	454.153
601 → 700	507.441	504.579	501.947
701 → 800	509.251	506.973	504.709
801 → 900	454.590	451.576	448.683
901 → 1000	464.702	462.129	459.560
Total	4738.411	4709.032	4681.560
Average Annual Cost	4.738	4.709	4.682

Table 5.22: Northern System Pumping Costs for Various South Para Nominal Winter Minimum Operating Levels - (2)

There were no failure occurrences for the any reservoir of the northern system for each of the South Para nominal winter minimum storage levels examined.

Consideration of the results shown in Tables 5.21 and 5.22, and plotted in Figure 5.6, highlight that significant potential reductions in operating costs can be achieved. Lowering the South Para Reservoir nominal winter minimum operating levels does not reduce the system reliability below a 99.9% level.

The lowest combined storage level reached in the South Para system (comprising the Warren, South Para and Barossa Reservoirs) during the 1000 year simulation of the system using a nominal winter minimum operating level of 2100 ML for South Para Reservoir was 6506 ML. This represents 2300 ML above the physical minimum operating level (more than 2 weeks demand) for every month in the year for the system. Using a nominal winter minimum operating level of 4100 ML for South Para Reservoir, the lowest combined storage level reached in the South Para system rises to 8438 ML representing 4200 ML above the physical minimum operating level (more than 4 weeks demand).

The South Para system can be supplemented by water pumped via the Swan Reach-Stockwell and the Mannum-Adelaide pumping systems. Before firm recommendations can be made regarding the nominal winter minimum operating level for South Para Reservoir, the impact of pumping system failures on the performance of the system should be assessed. The impact of the reliability of these pumping systems is considered later in this chapter in Section 5.3.

5.2.2.4 Northern System - Little Para and Millbrook Nominal Winter Minimum Operating Level Comparison

In this section, a comparison is made by modifying the operating rules associated with the Little Para and Millbrook Reservoirs of the northern system.

Variation of the Millbrook nominal winter minimum operating level will impact on the behaviour of the three reservoirs (Millbrook, Kangaroo Creek and Hope Valley) in the Torrens system as these three reservoirs are closely linked in the operation of the northern system. The nominal winter minimum operating level component of the target storage levels for the Little Para and Millbrook Reservoirs have been varied and a synthetic inflow and demand data record of 1000 years has been used to examine the effect on the long term operating behaviour for the system. For each of the nominal winter minimum operating level components of the target storage levels, 4 weeks of demand storage has been used to obtain the monthly target storage levels. For the three cases considered, the 70% inflow exceedance data set for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. For all other reservoirs, the nominal minimum operating level components of the target storage levels used in this analysis are those presented in Table B.14. Other details used in the HOMA model are given in Appendix B.

The comparative pumping costs obtained from the simulations are presented in Table 5.23 and the corresponding number of failures in Little Para Reservoir shown in Table 5.24.

Reducing the nominal winter minimum operating levels in both Millbrook and Little Para Reservoirs by 1000 ML has no apparent impact on the reliability of the northern system. If these minimum levels are reduced a further 1000 ML , failures in the Little Para Reservoir are induced. Both the Torrens and Little Para systems are highly dependent on the Mannum-Adelaide pumping system and the Millbrook pump station. It is important therefore to consider the impact of potential failures of these pumping systems on the reliability-cost tradeoffs for the operation of the northern system before recommendations can be made regarding the nominal winter minimum operating levels for the Millbrook and Little Para Reservoirs.

Water Year Range	Pumping Electricity Cost ($M)		
	3000 ML Millbrook 5000 ML Little Para Winter Mins.	2000 ML Millbrook 4000 ML Little Para Winter Mins.	1000 ML Millbrook 3000 ML Little Para Winter Mins.
1 → 100	455.799	451.150	446.688
101 → 200	446.163	441.643	437.229
201 → 300	517.482	513.776	510.192
301 → 400	482.386	478.505	475.498
401 → 500	482.606	478.439	474.364
501 → 600	469.082	465.001	461.308
601 → 700	514.964	511.237	507.597
701 → 800	516.579	513.441	509.459
801 → 900	463.493	459.375	455.379
901 → 1000	472.244	469.076	465.406
Total	4820.798	4781.643	4743.120
Average Annual Cost	4.821	4.782	4.743

Table 5.23: Northern System Pumping Costs for Various Little Para and Millbrook Nominal Winter Minimum Operating Levels

Water Year Range	Little Para Failure Occurrence					
	3000 ML Millbrook 5000 ML Little Para Winter Min.		2000 ML Millbrook 4000 ML Little Para Winter Min.		1000 ML Millbrook 3000 ML Little Para Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	3	3
101 → 200	0	0	0	0	14	6
201 → 300	0	0	0	0	25	12
301 → 400	0	0	0	0	8	5
401 → 500	0	0	0	0	11	6
501 → 600	0	0	0	0	13	6
601 → 700	0	0	0	0	22	9
701 → 800	0	0	0	0	19	11
801 → 900	0	0	0	0	19	7
901 → 1000	0	0	0	0	6	2
Total	0	0	0	0	140	55
Average Annual Cost	0.000	0.000	0.000	0.000	0.140	0.055

Table 5.24: Northern System Failure Occurrences for Various Little Para and Millbrook Nominal Winter Minimum Operating Levels

Potential does exist to reduce the nominal winter minimum operating levels for both the Little Para and Millbrook Reservoirs with no apparent reduction in reliability in terms of hydrologic risks related to inflows and demands with an associated reduction in average annual operating cost of approximately $39,000.

In conclusion, consideration of the reliability-cost tradeoffs for the northern Adelaide water supply system taking into account hydrologic variability, has revealed the potential for significant reductions in operating costs with little or no apparent loss in system reliability. The dependence of the northern system on pumping from the River Murray via the Mannum-Adelaide and the Swan Reach-Stockwell pumping systems together with the transfer facility at the Millbrook pumping station, necessitates the inclusion of the impact of the reliability of these pumping systems in the examination of reliability-cost tradeoffs. These impacts are considered later in this chapter in Section 5.3.

5.2.3 Synthetic Inflow Data Parameter Uncertainty Analysis

The generation of synthetic inflow data for the metropolitan Adelaide water supply system has involved a set of 1080 parameters representing the monthly statistical properties of the five streamflow gauging sites together with the coefficients of the $[A]$ and $[B]$ matrices. Details of the model used to generate this data has been presented in Section 4.4 of Chapter 4.

As discussed in Section 3.3 of Chapter 3, work by a number of researchers has highlighted the need to consider parameter uncertainty in the generation of synthetic streamflow data. This parameter uncertainty should be considered in the assessment of the expected performance of urban water supply headworks systems as the parameters are estimated from a limited length of historical record.

A rigorous analysis of all parameters is beyond the scope of the current study, however the examination of the effect of parameter uncertainty is still considered appropriate. Results for the southern system indicate that the Myponga Reservoir is the critical element in the metropolitan Adelaide system. In this section, parameter uncertainty for the inflows to this reservoir have been examined.

The 24 parameters that have the greatest impact on the Myponga Reservoir are the 12 monthly means and variances for the Myponga catchment. The values determined for these parameters by Baker and Dandy [10] using 42 years of historical data have been previously presented in Table 4.14 of Chapter 4. These values are also presented in Table 5.25.

Month	Mean Inflow (ML)	Standard Deviation (ML)
July	4391	3685
August	4480	3213
September	3054	3082
October	1479	1395
November	580	353
December	320	165
January	301	331
February	305	506
March	255	145
April	405	243
May	998	1045
June	2594	2803

Table 5.25: Myponga Historical Inflow Statistics

For the generation of synthetic data it is preferable to have normally distributed data. Using the raw historical data, parameters have been determined by Baker and Dandy to transform the historical data into a de-trended, deseasonalised, zero mean, unit variance, normally distributed set. The historical data has been transformed using a three parameter log-transform and the parameters obtained are presented in Table 5.26. For the month of December no transformation was required as the historical data was normally distributed. Further details of the procedures used to transform the data are presented in Baker [10].

Month	Mean of Transformed Data	Standard Deviation of Transformed Data	Shifting Parameter
July	8.16	0.79	-293
August	8.65	0.50	-1937
September	7.53	1.00	+98
October	6.92	0.88	+14
November	6.30	0.54	-48
December	-	-	normal
January	5.67	0.64	-59
February	5.44	0.75	-19
March	6.26	0.27	-284
April	6.18	0.43	-125
May	6.19	1.06	+156
June	7.27	1.10	+105

Table 5.26: Statistics of Myponga Transformed Inflow Data

Preliminary work by Crawley and Dandy [49] identified the critical months for the operation of the southern system as April, May and June. During these months the probability of failure occurring in the Myponga Reservoir is greatest. It was considered overly conservative to assume that inflows into Myponga Reservoir had been overestimated for all months in the year and therefore statistical parameters for only these three months have been varied. Using the modified inflow record for Myponga Reservoir the impact on the reliability of the southern system has been examined.

Estimates for the standard error of the monthly mean and standard deviation are given by Equations 5.1 and 5.2 (McMahon and Mein [203]).

$$\text{Standard error of mean} = \frac{s}{\sqrt{n}} \tag{5.1}$$

$$\text{Standard error of standard deviation} = \frac{s}{\sqrt{2n}} \tag{5.2}$$

where,

$$\begin{aligned} s &= \text{Standard deviation of the data} \\ n &= \text{Number of observations} \\ &= \text{(42 years for Myponga)} \end{aligned}$$

The proposed variations to the mean and standard deviation values for the months of April, May and June are to reduce the mean of the transformed data by 1.65 times the standard error of the mean and to increase the standard deviation of the transformed data by 1.65 times the standard error of the standard deviation. The factor 1.65 was selected as this represents the ninety fifth percentile for the estimated parameter assuming that it is approximately normally distributed. Cross-correlations between errors in the estimated monthly means and standard deviations are ignored in this simplified analysis. Low flow events have been identified as critical to the operation of the Myponga Reservoir. Reducing the mean and increasing the standard deviation of the transformed data will result in a greater number of these low flow events in the generated synthetic record and hence produce the 'worst-case' result. The modified means and standard deviations are presented in Table 5.27. The shifting parameters have not been altered.

Using these modified parameters, the previously generated synthetic data has been adjusted for the southern system. This data has been used as input to

Month	Mean of Transformed Data	Standard Deviation of Transformed Data	Shifting Parameter
July	8.16	0.79	-293
August	8.65	0.50	-1937
September	7.53	1.00	+98
October	6.92	0.88	+14
November	6.30	0.54	-48
December	-	-	normal
January	5.67	0.64	-59
February	5.44	0.75	-19
March	6.26	0.27	-284
April	6.07	0.51	-125
May	5.92	1.25	+156
June	6.99	1.30	+105

Table 5.27: Statistics of Myponga Transformed Inflow Data after Modification

HOMA to simulate the operation of the southern portion of the Adelaide headworks system. Reliability-cost tradeoffs have been examined for the operation of the southern water supply system by varying the Myponga nominal winter minimum operating level. In this examination, 70% exceedance inflow forecast sets have been used for all reservoirs in conjunction with the demand forecast set given in Table 4.11. The demand component of the target storage levels in all reservoirs in the system has been set at '4 weeks demand' as given in Table B.7. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA simulation model are given in Appendix B.

Pumping electricity costs obtained for the southern system using the range of operating rule sets are presented in Tables 5.28 and 5.29. The associated Myponga Reservoir failure occurrences are presented in Tables 5.30 and 5.31.

Water Year Range	Pumping Electricity Cost ($M)		
	11500 ML Winter Min.	10500 ML Winter Min.	9500 ML Winter Min.
1 → 100	154.520	153.736	152.945
101 → 200	162.761	161.964	161.200
201 → 300	192.546	191.548	190.555
301 → 400	166.686	165.771	164.957
401 → 500	182.329	181.454	180.669
501 → 600	154.727	153.952	153.287
601 → 700	185.639	184.625	183.628
701 → 800	182.536	181.732	180.956
801 → 900	162.428	161.506	160.678
901 → 1000	169.261	168.509	167.888
1001 → 1100	200.582	199.344	198.248
1101 → 1200	192.522	191.615	190.721
1201 → 1300	166.085	165.080	164.284
1301 → 1400	175.477	173.043	174.262
1401 → 1500	166.021	165.079	164.258
1501 → 1600	180.190	179.276	178.483
1601 → 1700	171.156	170.280	169.444
1701 → 1800	181.407	180.375	176.982
1801 → 1900	187.725	187.004	186.222
1901 → 2000	155.723	149.908	154.207
Total	3490.321	3465.801	3453.874
Average Annual Cost	1.745	1.733	1.727

Table 5.28: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels using Modified Myponga Inflow Data - (1)

Water Year Range	Pumping Electricity Cost ($M)		
	8500 ML Winter Min.	7500 ML Winter Min.	6500 ML Winter Min.
1 → 100	152.375	151.916	151.538
101 → 200	160.548	159.830	159.314
201 → 300	189.631	188.823	188.070
301 → 400	164.128	163.367	162.756
401 → 500	179.886	179.240	178.776
501 → 600	152.741	152.299	151.950
601 → 700	182.674	181.804	180.073
701 → 800	180.317	179.707	179.130
801 → 900	159.912	159.176	158.660
901 → 1000	167.285	166.754	166.229
1001 → 1100	197.310	196.633	195.929
1101 → 1200	189.875	189.050	188.500
1201 → 1300	163.595	163.029	162.584
1301 → 1400	173.768	173.255	172.785
1401 → 1500	163.489	162.855	162.220
1501 → 1600	177.670	176.949	176.244
1601 → 1700	168.720	168.053	167.454
1701 → 1800	178.556	177.860	177.279
1801 → 1900	185.514	184.823	184.074
1901 → 8000	153.565	152.935	152.405
Total	3441.559	3428.358	3416.970
Average Annual Cost	1.721	1.714	1.708

Table 5.29: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels using Modified Myponga Inflow Data - (2)

Water Year Range	Myponga Failure Occurrence					
	11500 ML Winter Min.		10500 ML Winter Min.		9500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0
101 → 200	0	0	0	0	1	1
201 → 300	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0
601 → 700	2	1	4	1	6	2
701 → 800	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0
901 → 1000	0	0	0	0	1	1
1001 → 1100	0	0	0	0	0	0
1101 → 1200	0	0	4	1	9	3
1201 → 1300	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0
1401 → 1500	0	0	2	1	5	2
1501 → 1600	0	0	0	0	1	1
1601 → 1700	0	0	0	0	2	1
1701 → 1800	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0
Total	2	1	10	3	25	11
Average Annual Failures	0.001	0.0005	0.005	0.0015	0.0125	0.0055
Lowest Myponga Reservoir Level attained (ML)	4091		3306		2374	

Table 5.30: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels using Modified Myponga Inflow Data - (1)

Water Year Range	Myponga Failure Occurrence					
	8500 ML Winter Min.		7500 ML Winter Min.		6500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	10	5
101 → 200	4	2	8	3	15	4
201 → 300	1	1	13	6	28	10
301 → 400	4	3	10	4	19	5
401 → 500	0	0	11	5	26	8
501 → 600	2	1	5	2	10	3
601 → 700	8	2	23	10	54	15
701 → 800	3	1	13	5	48	13
801 → 900	0	0	5	3	10	4
901 → 1000	4	1	6	2	16	5
1001 → 1100	8	3	15	6	29	8
1101 → 1200	31	7	62	15	104	20
1201 → 1300	1	1	6	3	16	6
1301 → 1400	3	1	9	3	18	5
1401 → 1500	8	2	14	3	22	4
1501 → 1600	7	3	23	6	36	7
1601 → 1700	6	4	19	6	32	8
1701 → 1800	6	4	31	11	67	17
1801 → 1900	6	3	18	7	41	10
1901 → 2000	0	0	2	1	8	3
Total	102	39	311	101	609	160
Average Annual Failures	0.051	0.0195	0.1555	0.0505	0.3045	0.080
Lowest Myponga Reservoir Level attained (ML)	1651		1268		573	

Table 5.31: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels using Modified Myponga Inflow Data - (2)

Results from Tables 5.28, 5.30, 5.29 and 5.31 together with the results from the unmodified Myponga inflow comparison previously presented in Tables 5.10, 5.12, 5.11 and 5.13 are summarised in Table 5.32.

Myponga Nominal Winter Minimum Operating Level (ML)	Original Myponga Inflow Statistics		Modified Myponga Inflow Statistics		Increase in Average Annual number of Failure Events
	Average Annual Pumping Costs ($M)	Average Annual Failure Events	Average Annual Pumping Costs ($M)	Average Annual Failure Events	
11500	1.730	0.0005	1.745	0.0005	0.0000
10500	1.721	0.0010	1.733	0.0015	0.0005
9500	1.714	0.0025	1.727	0.0055	0.0030
8500	1.707	0.0160	1.721	0.0195	0.0035
7500	1.701	0.0355	1.714	0.0505	0.0150
6500	1.695	0.0675	1.708	0.0800	0.0125

Table 5.32: Original and Modified Myponga Inflow - Annual Cost and Failure Summary

A number of observations can be drawn from the results presented in Table 5.32.

1. The results indicate that the effect of uncertainty of the inflow parameters will have some impact on the assessed reliability-cost tradeoffs for the system and the effect of this parameter uncertainty must be considered in the selection of an appropriate level of system reliability.

2. The largest increases in the average annual number of failure events occurs with the use of 7500 ML and 6500 ML nominal winter minimum operating levels for Myponga Reservoir. For the higher nominal winter minimum operating levels of 8500 ML to 11500 ML , the increase is relatively small, however still significant relative to the average annual failure event frequency determined using the original Myponga Reservoir

inflow statistics. If the required reliability for the southern system is for the failure frequency to be less than or equal to 1 in 400 years, it may be considered appropriate to select a Myponga nominal winter minimum operating level of 10500 ML rather than 9500 ML to accommodate the effect of the inflow parameter uncertainty.

3. A reduction in the mean inflow for the months of April, May and June results in an increase in the average annual pumping cost for the system. This increase in pumping cost is relatively consistent across the range of operating rules considered.

5.2.4 Summary

In this section, reliability-cost tradeoffs have been considered for the southern and northern components of the Adelaide water supply headworks system, taking into account potential variations in inflow and demand on the system. A range of operating rule sets have been considered and the pumping costs and failure occurrences associated with these operating rules determined. In the examination of the system, the bulk water transfer systems have been assumed to be fully available whenever required.

Results presented in this section highlight that the most sensitive component of the headworks system is Myponga Reservoir. Myponga Reservoir is also the only reservoir in the system that cannot be supplemented with water from the River Murray.

Potential exists in the results presented for significant reductions in system operating costs, with only small changes in system reliability. Because of the high dependence of the Adelaide system on the transfer of water from the River Murray, it is vital that the impact of the reliability of the bulk water transfer systems be considered in the overall reliability-cost assessment of the

Adelaide system. This assessment is undertaken in the following section.

5.3 Hydrologic and Component Reliability Assessment

This section considers both the hydrologic and bulk water transfer reliability factors that affect the reliability-cost tradeoffs for the metropolitan Adelaide water supply system. Using synthetic data generated for reservoir inflows and system demands, in conjunction with synthetic failure data for the bulk water transfer systems, the southern and northern components of the system are examined using the simulation/optimisation model HOMA.

The parameters determined for the bulk water transfer component reliabilities have a degree of uncertainty associated with them. In order to examine the sensitivity of the results to these uncertainties, variations to these parameters have been considered, and the effect on the simulation results for a range of operating rule sets presented.

5.3.1 Southern System

In the first assessment of the southern system, a demand storage level comparison previously undertaken is repeated with the inclusion of pumping system failures to examine the effect of these failures on the reliability-cost tradeoffs for the system. In the second assessment, the nominal winter minimum operating level for the Myponga Reservoir is varied and the effect of pumping system failures on reliability-cost tradeoffs examined.

In both these comparisons the physical minimum operating levels at which failure is assumed to occur in the southern system reservoirs are those previously given in Table 5.1.

5.3.1.1 Southern System - Demand Storage Level Comparison

In this section, a comparison is made between a range of operating rules for the southern system using runs with 8, 6, 4 and 2 weeks of demand storage components of the target storage levels. The nominal minimum operating levels components of the target storage levels used in this analysis are presented in Table B.4 of Appendix B. The performance of these operating rules are considered using 2000 years of synthetic inflow and demand data in conjunction with Monte Carlo failure data for the Murray Bridge-Onkaparinga pumping system.

Additional operating rules used in this comparison include the demand forecast set presented in Table 4.11 and the 70% exceedance forecast inflow sets presented in Tables 4.3 and 4.4. Other details used in the HOMA simulation model are given in Appendix B.

The streamflow and demand data has been generated using the models previously described in Chapter 4. The pumping system failure data has been generated using the model described in Section 4.6.4 of this same chapter.

The pumping costs obtained from the four simulations are presented in Table 5.33. The corresponding number of months of failure and failure events for the Myponga Reservoir are given in Table 5.34. This is identical to Table 5.8 which ignored the reliability of the bulk water transfer system. Again, no failures occurred in the Onkaparinga system. In all four simulations, there were two months when the pumping capacity was reduced to zero for the month. During these two months the online demand by small towns on the Murray Bridge-Onkaparinga pipeline was unable to be met. It has been assumed that during these two months, water would have been tankered to meet these supply requirements. An estimated cost for tankered water, used in this comparison is $10,000/ML. This cost has been taken from work presented by Dandy [61].

The results again highlight that supply failure from the Myponga Reservoir due to low inflow events is the critical consideration in the operation of the southern system. The Onkaparinga system comprising Mount Bold and Happy Valley Reservoirs are not critical in the operation of the system because of the large capacity of the Murray Bridge-Onkaparinga pumping system relative to the southern system demand.

For the range of operating rules considered and the pumping system reliability parameters assumed, pumping system failures have little impact on the overall reliability of the southern system.

Comparison of the pumping costs presented in Table 5.33 with those previously presented in Table 5.8 where perfect pumping reliability was assumed, reveals that the impact of pumping system failures is to marginally increase the overall pumping costs. This result is intuitive since, in addition to the costs associated with tankering to meet on-line demands, 'catch up' pumping will need to be undertaken at higher marginal pumping costs in subsequent months following a pumping system failure.

The results also show that using a demand storage component of '8 weeks demand' as part of the target storage levels in the operation of the southern system, a failure of supply from Myponga Reservoir would occur with a frequency of approximately 1 in 400 years (99.75% reliability) and the failure would have an average duration of approximately 3 months.

Results from this assessment of the southern system highlight that the operating rules could be modified from using an '8 weeks demand' component of the target storage levels to a '4 weeks demand' component with no apparent reduction in system reliability. These changes in operating rules would result in

Water Year Range	Pumping Electricity Cost ($M)			
	8 Weeks Demand Target Storages	6 Weeks Demand Target Storages	4 Weeks Demand Target Storages	2 Weeks Demand Target Storages
1 → 100	167.511	157.519	152.417	145.826
101 → 200	175.513	166.277	160.650	153.611
201 → 300	202.897	194.484	189.804	183.444
301 → 400	176.999	168.678	164.166	158.053
401 → 500	191.647	184.646	180.288	173.714
501 → 600	167.207	157.705	152.849	147.020
601 → 700	196.957	187.863	182.644	176.251
701 → 800	193.291	184.459	180.306	174.158
801 → 900	174.786	165.553	160.000	153.043
901 → 1000	181.841	172.088	167.571	161.009
1001 → 1100	210.734	202.419	197.885	191.104
1101 → 1200	203.301	194.838	190.229	183.692
1201 → 1300	176.675	168.472	163.526	157.762
1301 → 1400	187.187	178.386	173.934	167.835
1401 → 1500	177.890	168.534	163.712	157.543
1501 → 1600	191.463	182.491	177.892	171.550
1601 → 1700	182.604	173.397	168.947	162.935
1701 → 1800	192.013	182.153	178.649	172.696
1801 → 1900	198.016	189.941	185.451	179.801
1901 → 2000	167.713	158.478	153.882	147.386
Sub Total	3716.245	3538.381	3444.802	3319.433
Online Supply Tankering Cost	14.850	14.850	14.850	14.850
Total	3731.095	3553.231	3459.652	3334.283
Average Annual Cost	1.866	1.777	1.730	1.667

Table 5.33: Southern System Pumping Costs for Various Demand Storage Levels

Water Year Range	Myponga Failure Occurrence							
	8 Weeks Demand Target Storage		6 Weeks Demand Target Storage		4 Weeks Demand Target Storage		2 Weeks Demand Target Storage	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0	0	0
101 → 200	0	0	0	0	0	0	1	1
201 → 300	0	0	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0	0	0
601 → 700	3	1	3	1	3	1	3	1
701 → 800	0	0	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0	0	0
901 → 1000	0	0	0	0	0	0	0	0
1001 → 1100	0	0	0	0	0	0	0	0
1101 → 1200	6	2	6	2	6	2	6	2
1201 → 1300	0	0	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0	0	0
1401 → 1500	3	1	3	1	3	1	4	2
1501 → 1600	0	0	0	0	0	0	1	1
1601 → 1700	2	1	2	1	2	1	2	1
1701 → 1800	0	0	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0	0	0
Total	14	5	14	5	14	5	17	8
Average Annual Failures	0.007	0.003	0.007	0.003	0.007	0.003	0.009	0.004

Table 5.34: Myponga Failure Occurrences for Various Demand Storage Levels

an average reduction in operating costs of the order of approximately $140,000 per annum.

5.3.1.2 Southern System - Myponga Nominal Winter Minimum Operating Level Comparison

In this comparison, the Myponga Reservoir nominal winter minimum operating level has been varied and the reliability-cost tradeoffs for the southern system examined using 2000 years of synthetic inflow and demand data in conjunction with Monte Carlo failure data for the Murray Bridge-Onkaparinga pumping system. The 70% inflow exceedance sets given in Tables 4.3 and 4.4 and the '4 week demand' component of the target storages given in Table B.7 have been used for all runs. The demand forecast set adopted in this comparison is given in Table 4.11. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA simulation model are given in Appendix B.

The streamflow and demand data has been generated using the models previously described in Chapter 4. The pumping system failure data has been generated using the model described in Section 4.6.4 of this same chapter.

The comparative pumping costs obtained for each of the operating rule sets considered are given in Tables 5.35 and 5.36. In all six simulations, there were two months when the pumping capacity was reduced to zero for the month. During these two months the online demand by small towns on the Murray Bridge-Onkaparinga pipeline was unable to be met. It has been assumed that during these two months, water would have been tankered to meet these supply requirements. An estimated cost for tankered water, used in this comparison is $10,000/ML. The corresponding failure occurrences in Myponga Reservoir are shown in Tables 5.37 and 5.38. For each of the operating rule sets considered, no failures occurred in the Onkaparinga system.

Water Year Range	Pumping Electricity Cost ($M)		
	11500 ML Winter Min.	10500 ML Winter Min.	9500 ML Winter Min.
1 → 100	153.845	153.065	152.417
101 → 200	162.125	161.393	160.650
201 → 300	191.745	190.723	189.804
301 → 400	165.936	165.070	164.166
401 → 500	181.904	181.085	180.288
501 → 600	154.246	153.446	152.849
601 → 700	184.623	183.595	182.644
701 → 800	181.745	180.993	180.306
801 → 900	161.613	160.840	160.000
901 → 1000	168.876	168.203	167.571
1001 → 1100	200.035	198.863	197.885
1101 → 1200	191.931	191.069	190.229
1201 → 1300	165.114	164.246	163.526
1301 → 1400	175.128	174.515	173.934
1401 → 1500	165.350	164.486	163.712
1501 → 1600	179.535	178.699	177.892
1601 → 1700	170.547	169.726	168.947
1701 → 1800	180.562	179.561	178.649
1801 → 1900	186.942	186.125	185.451
1901 → 2000	155.358	154.592	153.882
Sub Total	3477.716	3460.295	3444.802
Online Supply Tankering Cost	14.850	14.850	14.850
Total	3492.566	3475.145	3459.652
Average Annual Cost	1.746	1.738	1.730

Table 5.35: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels - (1)

Water Year Range	Pumping Electricity Cost ($M) 8500 ML Winter Min.	7500 ML Winter Min.	6500 ML Winter Min.
1 → 100	151.877	151.465	151.200
101 → 200	160.002	159.486	159.054
201 → 300	188.953	188.210	187.637
301 → 400	163.389	162.821	162.238
401 → 500	179.641	179.175	178.720
501 → 600	152.376	151.990	151.608
601 → 700	181.764	181.079	180.401
701 → 800	179.640	179.073	178.563
801 → 900	159.294	158.682	158.223
901 → 1000	167.058	166.502	165.965
1001 → 1100	197.043	196.365	195.744
1101 → 1200	189.439	188.754	188.191
1201 → 1300	163.002	162.533	162.073
1301 → 1400	173.490	173.064	172.704
1401 → 1500	163.051	162.486	161.907
1501 → 1600	177.059	176.356	175.744
1601 → 1700	168.215	167.550	166.990
1701 → 1800	177.945	177.336	176.755
1801 → 1900	184.718	183.953	183.274
1901 → 8000	153.263	152.678	152.167
Sub Total	3431.219	3419.558	3409.158
Online Supply Tankering Cost	14.850	14.850	14.850
Total	3446.069	3434.408	3424.008
Average Annual Cost	1.723	1.717	1.712

Table 5.36: Southern System Pumping Costs for Various Myponga Nominal Winter Minimum Operating Levels - (2)

Water Year Range	Myponga Failure Occurrence					
	11500 ML Winter Min.		10500 ML Winter Min.		9500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
$1 \rightarrow 100$	0	0	0	0	0	0
$101 \rightarrow 200$	0	0	0	0	0	0
$201 \rightarrow 300$	0	0	0	0	0	0
$301 \rightarrow 400$	0	0	0	0	0	0
$401 \rightarrow 500$	0	0	0	0	0	0
$501 \rightarrow 600$	0	0	0	0	0	0
$601 \rightarrow 700$	1	1	2	1	3	1
$701 \rightarrow 800$	0	0	0	0	0	0
$801 \rightarrow 900$	0	0	0	0	0	0
$901 \rightarrow 1000$	0	0	0	0	0	0
$1001 \rightarrow 1100$	0	0	0	0	0	0
$1101 \rightarrow 1200$	0	0	3	1	6	2
$1201 \rightarrow 1300$	0	0	0	0	0	0
$1301 \rightarrow 1400$	0	0	0	0	0	0
$1401 \rightarrow 1500$	0	0	0	0	3	1
$1501 \rightarrow 1600$	0	0	0	0	0	0
$1601 \rightarrow 1700$	0	0	0	0	2	1
$1701 \rightarrow 1800$	0	0	0	0	0	0
$1801 \rightarrow 1900$	0	0	0	0	0	0
$1901 \rightarrow 2000$	0	0	0	0	0	0
Total	1	1	5	2	14	5
Average Annual Failures	0.0005	0.0005	0.0025	0.001	0.007	0.0025
Lowest Myponga Reservoir Level attained (ML)	4364		3884		3387	

Table 5.37: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels - (1)

Water Year Range	Myponga Failure Occurrence					
	8500 ML Winter Min.		7500 ML Winter Min.		6500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	8	5
101 → 200	2	1	7	3	12	4
201 → 300	1	1	8	3	25	9
301 → 400	2	2	9	4	13	4
401 → 500	0	0	6	4	18	7
501 → 600	2	1	3	1	8	2
601 → 700	7	2	10	3	33	15
701 → 800	3	1	10	4	36	11
801 → 900	0	0	0	0	2	1
901 → 1000	0	0	2	1	10	3
1001 → 1100	7	4	10	4	25	8
1101 → 1200	24	7	47	13	85	19
1201 → 1300	0	0	2	1	10	4
1301 → 1400	3	1	7	3	11	3
1401 → 1500	7	2	13	3	20	4
1501 → 1600	6	3	19	6	31	6
1601 → 1700	5	3	17	6	28	6
1701 → 1800	3	2	18	6	48	12
1801 → 1900	3	2	12	5	33	9
1901 → 2000	0	0	2	1	8	3
Total	75	32	202	71	464	135
Average Annual Failures	0.0375	0.016	0.101	0.0355	0.232	0.0675
Lowest Myponga Reservoir Level attained (ML)	2509		1872		970	

Table 5.38: Myponga Failure Occurrences for Various Myponga Nominal Winter Minimum Operating Levels - (2)

Comparison of the pumping costs presented in Tables 5.35 and 5.36 with those previously presented in Tables 5.10 and 5.11 where perfect pumping reliability was assumed, reveals that the impact of pumping system failures is to marginally increase the overall pumping costs. This increase in pumping cost is relatively uniform over the range of operating rules considered.

Comparison of the Myponga Reservoir failure occurrences presented in Tables 5.37 and 5.38 with those previously presented in Tables 5.12 and 5.13 reveals that no additional supply system failures resulted from the inclusion of pumping system failures in the simulation of the southern system. The impact of failures on the pumping system does not affect failures within Myponga Reservoir over the range of operating rules considered. This result highlights the capacity of the Murray Bridge-Onkaparinga pumping system relative to the southern system demand and the opportunity to modify the operating rules for this southern system, with only minor impact on the system reliability.

5.3.2 Northern System

In the first assessment of the northern system, a demand storage level comparison previously undertaken in Section 5.2 is repeated with the inclusion of pumping system failures to examine the effect of these failures on the reliability-cost tradeoffs for the system. In the second assessment, the nominal winter minimum operating level for South Para Reservoir is varied and the effect of pumping system failures on reliability-cost tradeoffs is determined. In the third assessment, the nominal winter minimum operating levels for the Millbrook and Little Para Reservoirs are varied and the effect of pumping system failures examined.

In all three comparisons, the physical minimum operating levels at which failure is assumed to occur in the northern system reservoirs are those previously given in Table 5.17.

5.3.2.1 Northern System - Demand Storage Level Comparison

In this section, a comparison is made between a range of operating rule sets for the northern system using simulations with 8, 6, 4 and 2 weeks of demand as the 'demand storage' component of the reservoir target storage levels. For each of the four cases considered, the 70% inflow exceedance data sets for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. These operating rule sets are examined using 1000 years of synthetic inflow and demand data in conjunction with Monte Carlo failure data for the Mannum-Adelaide and Swan Reach-Stockwell pumping systems and the Millbrook pump station. Details of the components comprising the target storage levels for the northern Adelaide system are presented in Appendix B in Tables B.15, B.17, B.19, B.21, B.16, B.18, B.20 and B.22. The nominal minimum operating levels components of the target storage levels used in this analysis are presented in Table B.14. Other details used in the HOMA simulation model are given in Appendix B.

This streamflow and demand data has been generated using the models previously described in Chapter 4. The pumping system failure data has been generated using the model described in Section 4.6.4 of this same chapter.

The comparative pumping costs are presented in Table 5.39. There were two months during the simulations of the northern system when failure of the Mannum-Adelaide pumping system resulted in the inability for on-line demands to be met. (Simulation of the Swan Reach-Stockwell pipeline does not include on-line demands). During these months it has been assumed that water would have been tankered to meet these supply requirements. An estimated cost for tankered water, used in this comparison is $10,000/ML. This cost has been taken from work presented by Dandy [61].

No system failures occurred within any of the 1000 year simulations for the northern system for the four sets of operating rules considered.

Comparison of the pumping costs presented in Table 5.39 with those previously presented in Table 5.20 reveals that the impact of the inclusion of pumping system failures on the operation of the system under the operating rules considered is to marginally increase the annual average pumping costs. This increase in pumping costs is relatively uniform across the four operating rules considered.

The results indicate that the current pumping capacity and its assessed reliability attributes, together with the nominal minimum storage levels in the northern system, are adequate to accommodate any hydrological shortfalls even when the demand storage component of the target storage values is reduced to two weeks of demand.

Water Year Range	Pumping Electricity Cost ($M)			
	8 Weeks Demand Target Storages	6 Weeks Demand Target Storages	4 Weeks Demand Target Storages	2 Weeks Demand Target Storages
1 → 100	480.325	465.230	457.227	446.463
101 → 200	470.142	456.109	447.460	435.944
201 → 300	542.259	527.875	518.912	510.491
301 → 400	506.467	492.807	484.377	474.379
401 → 500	508.102	493.326	483.195	473.018
501 → 600	493.659	479.914	470.551	460.326
601 → 700	539.775	524.695	516.338	507.304
701 → 800	541.131	525.914	518.293	509.402
801 → 900	485.956	473.450	464.498	455.159
901 → 1000	497.440	484.171	474.539	464.852
Sub Total	5065.256	4923.494	4835.390	4737.338
Online Supply Tankering Cost	6.770	6.770	6.770	6.770
Total	5072.026	4930.264	4842.160	4744.108
Average Annual Cost	5.072	4.930	4.842	4.744

Table 5.39: Northern System Pumping Costs for Various Demand Storage Levels

Potential exists for reductions in the northern system operating costs of up to 6% with no reduction in system reliability, taking into account both hydrologic factors and pumping system reliability. It is therefore recommended that the 'demand storage' component of the target storage levels forming a portion of the operating rules for the northern system, be reduced to 4 weeks demand.

5.3.2.2 Northern System - South Para Nominal Winter Minimum Operating Level Comparison

In this section, a comparison is made of a range of operating rule sets for the northern system involving the variation of the South Para nominal winter minimum operating level. Using these operating rule sets, reliability-cost tradeoffs are examined including the effect of hydrological variability and random failures of the three pumping systems utilised in the northern Adelaide water supply system.

A synthetic inflow, demand and pumping system failure data record of 1000 years has been used to examine the long term operating behaviour for the system under these operating rule sets. For each of the nominal winter minimum operating level components of the target storage levels considered, 4 weeks of demand storage has been used to obtain the monthly target storage levels. For each of the six cases considered, the 70% inflow exceedance data sets for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. For all other reservoirs, the nominal minimum operating level components of the target storage levels used in this analysis are those presented in Table B.14. Other details used in the HOMA model are given in Appendix B.

The comparative pumping costs are given in Tables 5.40 and 5.41. Online supply tankering costs are again included for the two months where failure of the Mannum-Adelaide pipeline resulted in insufficient monthly pumping system capacity to meet the online demands.

Water Year Range	Pumping Electricity Cost ($M) 11100 ML Winter Min.	10100 ML Winter Min.	8100 ML Winter Min.
1 → 100	457.527	456.041	452.718
101 → 200	447.460	445.320	441.605
201 → 300	518.912	517.415	514.323
301 → 400	484.377	482.626	479.330
401 → 500	483.915	482.208	478.064
501 → 600	470.551	468.411	464.764
601 → 700	516.338	515.040	511.683
701 → 800	518.293	516.701	512.668
801 → 900	464.498	462.770	459.278
901 → 1000	474.539	472.778	469.427
Sub Total	4835.390	4819.310	4783.860
Online Supply Tankering Cost	6.770	6.770	6.770
Total	4842.160	4826.080	4790.630
Average Annual Cost	4.842	4.826	4.791

Table 5.40: Northern System Pumping Costs for Various South Para Nominal Winter Minimum Operating Levels - (1)

There were no occurrences of failure in any reservoir during the simulations of the northern system for each of the South Para winter minimum storage levels examined. Consideration of the results presented in Tables 5.40 and 5.41 with the inclusion of pumping system failures in contrast to those presented in Tables 5.21 and 5.22, where 'perfect' operation of the pumping systems were assumed, highlights the impact of pumping system failures on the South Para system.

These failures result in a relatively uniform increase in the system operating costs over the range of operating rule sets considered. With the inclusion of pumping system failures, the lowest combined storage level reached in the South Para system (comprising the Warren, South Para and Barossa Reser-

Water Year Range	Pumping Electricity Cost ($M)		
	6100 ML Winter Min.	4100 ML Winter Min.	2100 ML Winter Min.
1 → 100	449.539	446.681	443.807
101 → 200	438.334	435.480	432.846
201 → 300	510.320	508.334	505.464
301 → 400	475.519	473.462	470.141
401 → 500	474.542	471.013	467.739
501 → 600	461.497	458.599	455.569
601 → 700	508.811	506.171	503.564
701 → 800	510.039	508.651	505.541
801 → 900	455.634	451.725	449.720
901 → 1000	466.054	463.486	460.891
Sub Total	4750.289	4723.602	4695.282
Online Supply Tankering Cost	6.770	6.770	6.770
Total	4757.059	4730.372	4702.052
Average Annual Cost	4.757	4.730	4.702

Table 5.41: Northern System Pumping Costs for Various South Para Nominal Winter Minimum Operating Levels - (2)

voirs) during the 1000 year simulation of the northern system, using a nominal winter minimum operating level of 2100 ML for the South Para Reservoir was 5232 ML. This represents 1000 ML above the physical minimum operating level for every month in the year for the system. Using a nominal minimum operating level of 4100 ML for the South Para Reservoir, the lowest combined storage level reached in the South Para system rises to 7283 ML representing 3100 ML above the physical minimum operating level (more than 2 weeks demand). Although these minimum levels are lower than those reached when 'perfect' pumping systems are assumed for the northern system, the South Para system storages still do not fall below the physical minimum operating level.

These results reveal that including the impact of the assessed pumping system reliability has only minimal effect on the operation of the northern system over the range of the South Para nominal winter minimum operating levels considered. Significant potential reductions in operating costs can be achieved by lowering these levels. Although the failure frequency is less than 1 in 1000 years (99.9%) for all operating rule sets considered, the lower minimum storage levels attained with the inclusion of pumping system failures highlight the need to consider the reliability of the pumping systems in reliability-cost tradeoff assessments.

It is recommended that the nominal winter minimum operating level for the South Para Reservoir be lowered from 11,100 ML to 2,100 ML with an associated reduction in average annual operating costs of $140,000.

5.3.2.3 Northern System - Little Para and Millbrook Nominal Winter Minimum Operating Level Comparison

In this section, a comparison is made of a range of operating rule sets for the northern system involving the variation of the Little Para and Millbrook nominal winter minimum operating levels. Variation of the Millbrook nominal winter minimum operating level will impact on the behaviour of the three reservoirs (Millbrook, Kangaroo Creek and Hope Valley) in the Torrens system as these three reservoir are closely linked in the operation of the northern system.

Using these operating rule sets, reliability-cost tradeoffs are examined including the effect of hydrological variability and random failures of the three pumping systems utilised in the northern Adelaide water supply system.

A synthetic inflow, demand and pumping system failure data record of 1000 years has been used to examine the long term operating behaviour for the system under these operating rule sets. For each of the nominal

The nominal winter minimum operating level component of the target storage levels for the Little Para and Millbrook Reservoirs have been varied and a synthetic inflow, demand and pumping system failure data record of 1000 years has been used to examine the effect on the long term operating behaviour for the system. For the three operating rule sets considered, the 70% exceedance inflow forecast sets, the forecast demand set presented in Table 4.12 and '4 weeks demand' storage have been used. For all other reservoirs, the nominal minimum operating level components of the target storage levels used in this analysis are those presented in Table B.14. Other details used in the HOMA model are given in Appendix B.

The comparative pumping costs are given in Table 5.42 and the number of failures in Little Para Reservoir presented in Table 5.43.

Water Year Range	Pumping Electricity Cost ($M)		
	3000 ML Millbrook 5000 ML Little Para Winter Mins.	2000 ML Millbrook 4000 ML Little Para Winter Mins.	1000 ML Millbrook 3000 ML Little Para Winter Mins.
1 → 100	457.227	452.755	448.256
101 → 200	447.460	442.799	438.330
201 → 300	518.912	514.304	511.516
301 → 400	484.377	480.392	476.527
401 → 500	483.915	479.613	475.495
501 → 600	470.551	466.353	462.558
601 → 700	516.338	511.944	508.834
701 → 800	518.293	514.211	511.926
801 → 900	464.498	460.393	455.551
901 → 1000	474.539	470.439	466.691
Sub Total	4835.390	4793.203	4755.684
Online Supply Tankering Cost	6.770	6.770	6.770
Total	4842.160	4799.973	4762.454
Average Annual Cost	4.842	4.800	4.762

Table 5.42: Northern System Pumping Costs for Various Little Para and Millbrook Nominal Winter Minimum Operating Levels

Water Year Range	Little Para Failure Occurrence					
	3000 ML Millbrook 5000 ML Little Para Winter Min.		2000 ML Millbrook 4000 ML Little Para Winter Min.		1000 ML Millbrook 3000 ML Little Para Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	3	3
101 → 200	0	0	0	0	15	7
201 → 300	0	0	0	0	27	12
301 → 400	0	0	0	0	8	5
401 → 500	0	0	0	0	12	5
501 → 600	0	0	0	0	13	6
601 → 700	0	0	0	0	22	9
701 → 800	0	0	0	0	19	11
801 → 900	0	0	0	0	19	9
901 → 1000	0	0	0	0	8	3
Total	0	0	0	0	146	70
Average Annual Failures	0.000	0.000	0.000	0.000	0.146	0.070

Table 5.43: Little Para Failure Occurrences for Various Little Para and Millbrook Nominal Winter Minimum Operating Levels

Consideration of the results presented in Tables 5.42 and 5.43, with the inclusion of the bulk water transfer failures, highlights the impact on the operation of the northern system.

The inclusion of pumping system failures again result in a relatively uniform increase in the system operating costs over the range of operating rule sets considered. Reducing the nominal winter minimum operating level in Little Para Reservoir to 4000 ML and in Millbrook Reservoir to 2000 ML has no apparent impact on the reliability of the system.

If these minimum levels are reduced a further 1000 ML, failures in the Little Para Reservoir are induced, but not in the Torrens system. Comparison of the failure occurrences in Table 5.43 with those previously presented in Table 5.24 reveals a slight increase in the frequency of failure for the same operating rule set with the inclusion of pumping system failures. A sensitivity analysis of the assumed pumping system critical component reliability attributes is considered later in this section.

Results presented in Tables 5.42 and 5.43 reveal that even with the inclusion of pumping system failures, potential exists for the reduction in both the Little Para and Millbrook Reservoir nominal winter minimum operating levels with only a small reduction in system reliability. Lowering the Little Para and Millbrook nominal winter minimum operating levels from 5000 ML and 3000 ML to 4000 ML and 2000 ML respectively, results in a reduction in average annual northern system operating costs of $42,000 while maintaining the system reliability at greater than 99.9%.

In conclusion, the impact of the inclusion of the reliability of the three major pumping systems in the assessment of northern system reliability-cost trade-offs, for the assessed pumping system reliability attributes and over the range of operating rule sets considered is small.

It is evident that even with the inclusion of the effect of pumping system failures, the northern system is extremely reliable. Significant potential has been identified for the reduction in system operating costs with minimal impact on the system reliability.

5.3.3 Component Parameter Uncertainty Analysis

In the estimation of the parameters associated with the component reliabilities for the bulk water transfer system, there will be an associated level of uncertainty. It is therefore important to consider the sensitivity of the results to these uncertainties.

Those components for which failure will lower the bulk water transfer system capacity to zero for one of the pumping systems, are likely to have the largest impact upon the performance of the system. If the mean repair time for these components has been underestimated or the component failure frequency has been overestimated, then the assessed long-term performance of the system will be non-conservative.

In this section, the effect of the parameter uncertainty will be examined for some of the components identified as having the greatest impact upon the water supply headworks system reliability-cost tradeoffs. Although a rigorous examination of all parameters has not been undertaken, the effect of certain key components will be considered for two of the three major pumping systems.

5.3.3.1 Murray Bridge-Onkaparinga Pumping System

Examination of Table 4.28 in Chapter 4 reveals that there are two critical components for which failure will bring the capacity of the Murray Bridge-Onkaparinga pumping system to zero and which also have the highest failure frequencies. These are the EWS main circuit breakers in each of the three pump stations and the external high voltage cables at the Murray Bridge-Onkaparinga no. 1 pump station. If the reliability parameters for these components have been underestimated, then the assessed long-term performance of the southern component of the water supply system will be assessed incorrectly.

In this section, the effect of modifying the assumed component reliability parameters for these two components will be considered. If the component parameters shown in Table 5.44 are assumed for Murray Bridge-Onkaparinga pumping system (in contrast to those previously adopted as shown in Table 4.28), and all other component parameters are held at their previously determined values, then the application of frequency-duration analysis to the overall pumping system produces the pumping system state transition attributes shown in Tables 5.45 and 5.46.

Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
EWS Main Circuit Breakers	14	1 in 20	0.998082
External High Voltage Cables	14	1 in 20	0.998082

Table 5.44: Modified Murray Bridge-Onkaparinga Pumping System Critical Component Reliability Data

Using these results, random failure events have been generated for the Murray Bridge-Onkaparinga pumping system using the Monte Carlo failure simulation model. This model requires the input of a 'seed' value to commence the generation of these random failure events. In order to ensure a realistic comparison, the same 'seed' value has been used for the generation of failure events with the modified pumping system reliability attributes as that used to generate the failure events with the original pumping system reliability attributes.

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1.000000	0.141554	0.000000	0.002885	0.960667
2.000000	0.147267	0.070626	0.001161	0.313542×10^{-1}
3.000000	0.000596	0.142647	0.000900	0.127799×10^{-3}
4.000000	0.002059	0.073258	0.000000	0.785058×10^{-2}
				1.000000

Table 5.45: Modified Murray Bridge-Onkaparinga Pumping System Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (ML /Month)
1.000000	0.277125×10^{-2}	346.654358	0.360847×10^{3}	15140
2.000000	0.225083×10^{-2}	13.930053	0.444280×10^{3}	10100
3.000000	0.183450×10^{-4}	6.966402	0.545107×10^{5}	5050
4.000000	0.575116×10^{-3}	13.650441	0.173878×10^{4}	0

Table 5.46: Modified Murray Bridge-Onkaparinga Pumping System Frequency Duration Analysis Results (2)

The failure events generated using the modified pumping system reliability attributes are of longer duration and will include some additional failure events that did not occur in the 'base' data generated. The southern component of the metropolitan Adelaide water supply system has then been simulated using the HOMA model with the synthetic inflow and demand data previously generated and the Monte Carlo pumping system failure data.

The following operating rule sets have been considered in the simulation of the southern system and a direct comparison can be made with results from a previous simulation of the system using the 'base' component reliability data previously presented in this section.

In these simulations the Myponga nominal winter minimum operating level has been considered over the range 11500 ML to 6500 ML and the reliability-cost tradeoffs examined using 2000 years of synthetic data. The 70% inflow exceedance set and a '4 week demand' component of the target storages for the southern system reservoirs have been used for all simulations undertaken. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA simulation model are given in Appendix B.

The pumping costs obtained from these simulations are presented in Tables 5.47 and 5.48 and the corresponding failure occurrences in the Myponga Reservoir are presented in Tables 5.49 and 5.50. No failures occurred in the Onkaparinga system. During each of the simulations of the southern system there were 28 months when the reduced pipeline pumping capacity resulted in the Murray Bridge-Onkaparinga online demands being unable to be supplied from the pipeline due to pumping system failures. The total of the online demands unable to be supplied was 18,694 ML. If these demands were met by tankered water at a cost of $10,000/ML. then an additional $M 186.94 should be added to the pumping electricity cost as shown in Tables 5.47 and 5.48.

Water Year Range	Pumping Electricity Cost ($M)		
	11500 ML Winter Min.	10500 ML Winter Min.	9500 ML Winter Min.
1 → 100	153.946	153.164	152.516
101 → 200	162.260	161.527	160.781
201 → 300	192.014	191.017	190.099
301 → 400	165.962	165.096	164.194
401 → 500	181.886	181.065	180.266
501 → 600	154.299	153.499	152.902
601 → 700	184.684	183.654	182.703
701 → 800	181.740	180.985	180.298
801 → 900	161.795	161.022	160.181
901 → 1000	169.042	168.370	167.738
1001 → 1100	200.501	199.327	198.343
1101 → 1200	191.948	191.082	190.244
1201 → 1300	165.178	164.309	163.587
1301 → 1400	175.171	174.557	173.977
1401 → 1500	165.527	163.116	163.890
1501 → 1600	179.634	178.798	177.991
1601 → 1700	170.724	169.905	169.126
1701 → 1800	180.587	179.588	178.677
1801 → 1900	187.014	186.194	185.524
1901 → 2000	155.966	155.197	154.476
Sub Total	3479.878	3461.472	3447.513
Online Supply Tankering Cost	186.940	186.940	186.940
Total	3666.818	3648.412	3634.453
Average Annual Cost	1.833	1.824	1.817

Table 5.47: Southern System - Component Reliability Sensitivity Analysis Pumping Costs - (1)

Water Year Range	Pumping Electricity Cost ($M) 8500 ML Winter Min.	7500 ML Winter Min.	6500 ML Winter Min.
1 → 100	151.977	151.565	151.300
101 → 200	160.121	159.596	159.237
201 → 300	189.249	188.522	187.952
301 → 400	163.418	162.841	162.318
401 → 500	179.619	179.151	178.698
501 → 600	152.429	152.043	151.661
601 → 700	181.823	181.136	180.458
701 → 800	179.633	179.066	178.557
801 → 900	159.476	158.856	158.393
901 → 1000	167.225	166.669	166.132
1001 → 1100	197.436	196.736	196.206
1101 → 1200	189.460	188.737	188.197
1201 → 1300	163.063	162.594	162.134
1301 → 1400	173.533	173.108	172.748
1401 → 1500	163.229	162.663	162.086
1501 → 1600	177.159	176.455	175.844
1601 → 1700	168.402	167.738	167.179
1701 → 1800	177.973	177.364	176.782
1801 → 1900	184.801	184.036	183.372
1901 → 8000	153.842	153.258	152.744
Sub Total	3433.868	3422.134	3412.998
Online Supply Tankering Cost	186.940	186.940	186.940
Total	3620.808	3609.074	3599.938
Average Annual Cost	1.810	1.805	1.800

Table 5.48: Southern System - Component Reliability Sensitivity Analysis Pumping Costs - (2)

Water Year Range	Myponga Failure Occurrence					
	11500 ML Winter Min.		10500 ML Winter Min.		9500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	0	0
101 → 200	0	0	0	0	0	0
201 → 300	0	0	0	0	0	0
301 → 400	0	0	0	0	0	0
401 → 500	0	0	0	0	0	0
501 → 600	0	0	0	0	0	0
601 → 700	1	1	2	1	3	1
701 → 800	0	0	0	0	0	0
801 → 900	0	0	0	0	0	0
901 → 1000	0	0	0	0	0	0
1001 → 1100	0	0	0	0	0	0
1101 → 1200	0	0	3	1	6	2
1201 → 1300	0	0	0	0	0	0
1301 → 1400	0	0	0	0	0	0
1401 → 1500	0	0	0	0	3	1
1501 → 1600	0	0	0	0	0	0
1601 → 1700	0	0	0	0	2	1
1701 → 1800	0	0	0	0	0	0
1801 → 1900	0	0	0	0	0	0
1901 → 2000	0	0	0	0	0	0
Total	1	1	5	2	14	5
Annual Average	0.0005	0.0005	0.0025	0.001	0.007	0.0025
Lowest Myponga Reservoir Level attained (ML)	4364		3884		3387	

Table 5.49: Southern System - Component Reliability Sensitivity Analysis Myponga Failure Comparison - (1)

Water Year Range	Myponga Failure Occurrence					
	8500 ML Winter Min.		7500 ML Winter Min.		6500 ML Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	8	5
101 → 200	2	1	7	3	12	4
201 → 300	1	1	8	3	25	9
301 → 400	2	2	9	4	13	4
401 → 500	0	0	6	4	18	7
501 → 600	2	1	3	1	8	2
601 → 700	7	2	10	3	33	15
701 → 800	3	1	10	4	36	11
801 → 900	0	0	0	0	2	1
901 → 1000	0	0	2	1	10	3
1001 → 1100	8	4	14	5	31	9
1101 → 1200	24	7	47	13	85	19
1201 → 1300	0	0	2	1	10	4
1301 → 1400	3	1	7	3	11	3
1401 → 1500	7	2	13	3	20	4
1501 → 1600	6	3	19	6	31	6
1601 → 1700	5	3	17	6	28	6
1701 → 1800	3	2	18	6	48	12
1801 → 1900	3	2	12	5	33	9
1901 → 2000	0	0	2	1	8	3
Total	76	32	206	72	470	136
Annual Average	0.038	0.016	0.103	0.036	0.235	0.068
Lowest Myponga Reservoir Level attained (ML)	2509		1872		970	

Table 5.50: Southern System - Component Reliability Sensitivity Analysis Myponga Failure Comparison - (2)

Results from Tables 5.28, 5.30, 5.29 and 5.31, together with the results using the 'base' component reliability parameters previously presented in Tables 5.10, 5.12, 5.11 and 5.13 have been summarised in Table 5.51.

Myponga Nominal Winter Minimum Operating Level (ML)	Original Murray Bridge -Onkaparinga Reliability Parameters		Modified Murray Bridge -Onkaparinga Reliability Parameters		Percentage Increase in Average Annual number of Failure Events (%)
	Average Annual Total Costs ($M)	Average Annual Failure Events	Average Annual Total Costs ($M)	Average Annual Failure Events	
11500	1.746	0.0005	1.833	0.0005	0
10500	1.738	0.0010	1.824	0.0010	0
9500	1.730	0.0025	1.817	0.0025	0
8500	1.723	0.0160	1.810	0.0160	0
7500	1.717	0.0355	1.805	0.0360	1.4
6500	1.712	0.0675	1.800	0.0680	0.7

Table 5.51: Original and Modified Murray Bridge-Onkaparinga Reliability Parameters - Annual Cost and Failure Summary

A number of observations can be drawn from the summary results presented in Table 5.51.

1. The effect of uncertainty of the pumping system reliability parameters has only minor impact on the assessed reliability-cost tradeoffs for the southern system for the range of operating rule sets considered.

2. A reduction in the assessed pumping system reliability for the southern system results in an increase in the average annual pumping cost for the system. This increase in pumping cost is relatively consistent across the range of operating rules considered.

3. The effect of parameter uncertainty of the pumping system reliability

attributes has only minor impact on the selection of operating rule sets from the range of an operating rule sets considered.

5.3.3.2 Mannum-Adelaide Pumping System

Examination of Table 4.29 in Chapter 4 reveals that the internal high voltage cables in each of the three Mannum-Adelaide pump stations and the intake valves at the Mannum-Adelaide no. 1 pump station have the highest failure frequencies of those components whose failure would lower the capacity of the Mannum-Adelaide pumping system to zero. Consideration of Tables 4.32, 4.34, 4.36 and 4.38 also reveals that the Mannum-Adelaide pumping system has the highest state probability for the zero capacity state of the four pumping systems.

If the reliability parameters for these components have been underestimated then the assessed long-term performance of the northern component of the water supply system will be assessed incorrectly.

In this section, the effect of modifying the assumed component reliability parameters for these components is considered. If the component parameters shown in Table 5.52 are assumed for the Mannum-Adelaide pumping system (in contrast to those originally determined and given in Table 4.29), and all other component parameters are held at their previously assumed values, then the application of frequency-duration analysis to the overall pumping system produces the pumping system state transition attributes shown in Tables 5.53 and 5.54.

Using these pumping system state transition attributes, random failure events have been generated for the Mannum-Adelaide pumping system using the Monte Carlo failure simulation model. The northern component of the metropolitan Adelaide water supply system has then been simulated with this failure

Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Intake Valves	14	1 in 10	0.996164
Internal High Voltage Cables	14	1 in 10	0.996164

Table 5.52: Modified Mannum-Adelaide Pumping System Critical Component Reliability Data

State	Rate of Entry	Rate of Departure (Up)	Rate of Departure (Down)	State Probability
1.000000	0.135557	0.000000	0.008555	0.905009
2.000000	0.154039	0.068966	0.003458	0.680592×10^{-1}
3.000000	0.214261	0.143845	0.003750	0.421415×10^{-4}
4.000000	0.000381	0.214169	0.003561	0.750195×10^{-7}
5.000000	0.013667	0.071601	0.000000	0.268895×10^{-1}
				1.000000

Table 5.53: Modified Mannum-Adelaide Pumping System Frequency Duration Analysis Results (1)

State	State Frequency	State Mean Duration (Days)	Cycle Time (Days)	Pumping Capacity (ML /Month)
1.000000	0.774259×10^{-2}	116.887093	0.129156×10^{3}	10408
2.000000	0.492913×10^{-2}	13.807547	0.202876×10^{3}	7806
3.000000	0.621987×10^{-5}	6.775300	0.160775×10^{6}	5204
4.000000	0.163340×10^{-7}	4.592857	0.612221×10^{8}	2602
5.000000	0.192533×10^{-2}	13.966190	0.519392×10^{3}	0

Table 5.54: Modified Mannum-Adelaide Pumping System Frequency Duration Analysis Results (2)

data in conjunction with the synthetic inflow and demand data previously generated.

The following operating rules have been considered in the simulation of the northern system and a comparison made with results from previous simulations of the system using the 'base' component reliability data previously presented in this section.

The operating rules considered for the northern system involve the nominal winter minimum operating level components of the target storage levels for the Little Para and Millbrook Reservoirs. These levels have been varied and a synthetic data record of 1000 years has been used to examine the effect on the long term operating behaviour for the system. For each of the nominal winter minimum operating level components of the target storage levels considered, 4 weeks of demand storage has been used to obtain the monthly target storage levels. The 70% inflow exceedance data sets for each reservoir in the system has been used together with the forecast demand set presented in Table 4.12. For all other reservoirs, the nominal minimum operating level components of the target storage levels used in this analysis are those presented in Table B.14. Other details used in the HOMA simulation model are given in Appendix B.

The operating rules for the Little Para and Millbrook Reservoirs have been selected from among the possible northern system operating rules previously considered, as variation of these operating rules has resulted in failure occurrence in the northern system. The purpose of this examination is to determine the additional impact of the pumping system component parameter uncertainty on the overall reliability-cost tradeoffs for the system.

The comparative pumping costs from these simulations of the northern system are given in Table 5.55 and the number of failures in the Torrens system and the Little Para Reservoir are shown in Tables 5.56 and 5.57.

In addition to the online supply tankering costs, there were several events when the combined failures of the Mannum-Adelaide pumping system, Millbrook pumping station and the storage level in Millbrook Reservoir resulted in insufficient capacity to meet supply requirements in the Anstey Hill demand zone. During these events, it has been assumed that the demand requirements were met by tankering water to Anstey Hill water filtration plant at a cost of $10,000/ML. These costs have been included within the results presented in Table 5.55.

Water Year Range	Pumping Electricity Cost ($M)		
	3000 ML Millbrook 5000 ML Little Para Winter Mins.	2000 ML Millbrook 4000 ML Little Para Winter Mins.	1000 ML Millbrook 3000 ML Little Para Winter Mins.
1 → 100	458.821	453.996	449.436
101 → 200	449.201	445.322	439.837
201 → 300	520.386	516.377	512.561
301 → 400	485.842	480.216	477.174
401 → 500	485.634	482.201	478.150
501 → 600	471.318	467.170	462.440
601 → 700	516.755	513.777	509.144
701 → 800	521.088	517.773	514.579
801 → 900	464.530	461.858	457.783
901 → 1000	474.194	471.055	467.474
Sub Total	4847.769	4809.745	4768.578
Online Supply Tankering Cost	141.930	141.930	141.930
Metropolitan Adelaide Supply Tankering Cost	39.860	48.660	120.070
Total	5029.559	5000.335	5030.578
Average Annual Cost	5.030	5.001	5.031

Table 5.55: Northern System - Component Reliability Sensitivity Analysis Pumping Costs

Water Year Range	Little Para Failure Occurrence					
	3000 ML Millbrook 5000 ML Little Para Winter Min.		2000 ML Millbrook 4000 ML Little Para Winter Min.		1000 ML Millbrook 3000 ML Little Para Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	6	4
101 → 200	0	0	0	0	16	7
201 → 300	0	0	0	0	29	14
301 → 400	0	0	0	0	8	5
401 → 500	0	0	0	0	14	5
501 → 600	0	0	0	0	19	9
601 → 700	2	1	2	1	25	12
701 → 800	0	0	0	0	21	13
801 → 900	0	0	0	0	20	10
901 → 1000	2	2	2	2	10	4
Total	4	3	4	3	168	83
Average Annual Failures	0.004	0.003	0.004	0.003	0.168	0.083

Table 5.56: Northern System - Component Reliability Sensitivity Analysis Little Para Failure Comparison

Water Year Range	Torrens System Failure Occurrence					
	3000 ML Millbrook 5000 ML Little Para Winter Min.		2000 ML Millbrook 4000 ML Little Para Winter Min.		1000 ML Millbrook 3000 ML Little Para Winter Min.	
	Mths.	Evts.	Mths.	Evts.	Mths.	Evts.
1 → 100	0	0	0	0	1	1
101 → 200	0	0	0	0	0	0
201 → 300	0	0	0	0	1	1
301 → 400	0	0	0	0	0	0
401 → 500	0	0	0	0	2	1
501 → 600	0	0	0	0	1	1
601 → 700	1	1	1	1	2	2
701 → 800	0	0	0	0	0	0
801 → 900	0	0	0	0	1	1
901 → 1000	0	0	0	0	3	2
Total	1	1	1	1	11	9
Average Annual Failures	0.001	0.001	0.001	0.001	0.011	0.009

Table 5.57: Northern System - Component Reliability Sensitivity Analysis Torrens System Failure Comparison

Results from Tables 5.55, 5.56 and 5.57, together with the results from the 'base' pumping system reliability comparison previously presented in Tables 5.42 and 5.43 have been summarised in Table 5.58.

Nominal Winter Minimum Operating Level ML.		Original Mannum -Adelaide Reliability Parameters		Modified Mannum -Adelaide Reliability Parameters		
Millbrook Reservoir	Little Para Reservoir	Average Annual Total Costs ($M)	Average Annual Little Para Failure Events	Average Annual Total Costs ($M)	Average Annual Little Para Failure Events	Average Annual Torrens System Failure Events
5000	3000	4.842	0.000	5.030	0.003	0.001
4000	2000	4.800	0.000	5.001	0.003	0.001
3000	1000	4.762	0.070	5.031	0.083	0.009

Table 5.58: Original and Modified Mannum-Adelaide Reliability Parameters - Annual Cost and Failure Summary

Comparison of the summarised results highlights the impact of the sensitivity associated with the assessed bulk water transfer system reliability data on the operation of the northern system.

It is evident that the northern system is more sensitive to uncertainty in the assessment of the bulk water transfer system reliability attributes than the southern system. Given the higher ratio of average annual pumping to annual pumping capacity for the northern system to the southern system, this result is not surprising.

When the Little Para and Millbrook nominal winter minimum operating levels are set at 1000 ML and 3000 ML respectively, the reduction in average annual pumping costs are outweighed by the increases in tankering costs to meet Anstey Hill demand requirements, resulting in a higher total cost than that obtained using Little Para and Millbrook nominal winter minimum operating levels of 2000 ML. and 4000 ML. respectively. These increases in tankering costs result from the increased volumes of water that can not be supplied through Anstey Hill as a result of insufficient capacity in Millbrook Reservoir in conjunction with failure of the Mannum-Adelaide pumping system. It is evident from these results that the level of storage held in Millbrook Reservoir will have significant impact on the northern system during Mannum-Adelaide pumping system failure events.

The results from this sensitivity analysis highlight the need to estimate the reliability attributes for the critical components in the bulk water transfer systems as accurately as possible.

5.3.4 Summary

In this section, the impact of both the hydrologic variability and the reliability of the bulk water transfer system on the reliability-cost tradeoffs for the operation of the southern and northern Adelaide water supply headworks systems has been examined.

Results presented in this section reveal that the impact of the reliability of the bulk water transfer system is generally limited over the range of operating rule sets considered. The results indicate that significant reductions in operating costs for both the southern and northern systems may be achieved with little reduction in the reliability of these systems.

Sensitivity analysis undertaken on the bulk water transfer system component reliabilities reveals that the northern system is more sensitive than the southern system to parameter variation.

In the selection of operating rule sets for the northern system, it may be prudent to adopt a slightly more conservative operating rule set because of the greater impact that uncertainty in the bulk water transfer system reliability parameters will have on the performance of the system.

5.4 Hydrologic and Component Reliability Assessment including Restriction Policies

This section considers the application of a restriction policy in addition to the hydrologic factors and the bulk water transfer reliability factors affecting the reliability-cost tradeoffs for the metropolitan Adelaide water supply system. Using a restriction policy together with a methodology for the assessment of economic losses resulting from the imposition of water restrictions, the overall reliability cost tradeoffs for the system have been examined.

In Sections 5.2 and 5.3 of this thesis, the Myponga Reservoir in the southern system has been identified as critical to the Adelaide water supply system. In this section, the southern system has been examined to identify the impact of the use of a water restriction policy on reliability-cost tradeoffs. In the first case study, a water restriction policy for the southern system has been considered where water restrictions are imposed on all demands on the southern system, in the event of the Myponga Reservoir falling below the restriction imposition target storage level. In the second case study, restrictions are imposed only on those demands that are directly met from the Myponga Reservoir.

5.4.1 Southern System Restriction Policy - Case 1

In this first case study, a range of operating rules have been considered and a restriction policy used to reduce demand from the full southern system when the monthly Myponga Reservoir storage level falls below a trigger target storage level. The imposition and relaxation trigger storage levels for Myponga Reservoir for each month of the year used in this case study are presented in Table 5.59.

The assumed reduction in outdoor water use associated with the three restriction classes is shown in Table 5.60 together with the economic costs associated with their imposition. These economic costs have been determined using the techniques described in Section 4.7 of Chapter 4.

Restriction Class	Imposition Trigger Storage in Myponga Reservoir (ML)	Relaxation Trigger Storage in Myponga Reservoir (ML)
Class 1	6000	6500
Class 2	5500	6000
Class 3	5000	5500

Table 5.59: Imposition and Relaxation Trigger Storage Levels for Myponga Reservoir

Restriction Class	Form of Restriction	Reduction in Outdoor Water Use (%)	Economic Restriction Costs ($M/Gl)
Class 1	Sprinklers banned 7 a.m. to 8 p.m.	20	1.03125
Class 2	No fixed sprinklers, hand hose any time	50	1.42194
Class 3	Hand hose 2 hours/day, pool filling and car washing restricted	80	2.55247

Table 5.60: Restriction Classes, Expected Reductions in Outdoor Water Use and Costs for the Southern System

In this first case study the Myponga nominal winter minimum operating level has been varied and the reliability-cost tradeoffs examined using 2000 years of synthetic inflow and demand data, in conjunction with Monte Carlo failure data for the Murray Bridge-Onkaparinga pumping system. The 70% inflow exceedance sets and a '4 week demand' component of the target storages for the southern system reservoirs have been used for each of the operating rule sets considered. The demand forecast set adopted in this comparison is given in Table 4.11. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA model are given in Appendix B.

In previous sections, the water filtration plant costs have not been presented since these costs are directly related to the volume of demands from the system. In the comparisons presented in this thesis the assumed Happy Valley and Myponga marginal water filtration plant costs are the same. Identical demands on the southern system, will therefore result in the same total water filtration plant costs no matter which reservoir is used to meet the demands. It has therefore not been necessary to include these costs in the reliability-cost tradeoffs for the system. With the application of water restrictions, it is appropriate to also include the reduction in water filtration plant costs in the overall reliability-cost tradeoffs. The unit costs for the two water filtration plants in the southern system are given in Appendix B.

Water supply costs for Myponga nominal winter minimum operating levels varying from 12500 ML to 6500 ML are presented in Tables 5.61, 5.62, 5.63, 5.64,5.65, 5.66 and 5.67. The number and level of restrictions imposed on the system for each of the operating rules considered are presented in Tables 5.68, 5.69, 5.70, 5.71, 5.72, 5.73 and 5.74.

With the application of the proposed water restriction policy, no failures occurred in Mount Bold, Happy Valley or Myponga Reservoirs for all operating rule sets considered except when the Myponga winter minimum was reduced to 6500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	154.733	0.000	0.000	0.000	154.733
101 → 200	162.979	8.380	0.000	0.000	171.359
201 → 300	192.746	0.000	0.000	0.000	192.746
301 → 400	166.860	6.470	0.000	0.000	173.330
401 → 500	182.830	0.000	0.000	0.000	182.830
501 → 600	154.913	0.000	0.000	0.000	154.913
601 → 700	185.620	0.000	3.202	0.092	188.730
701 → 800	182.585	0.000	0.000	0.000	182.585
801 → 900	162.523	0.000	0.000	0.000	162.523
901 → 1000	169.682	0.000	0.000	0.000	169.682
1001 → 1100	201.246	0.000	0.000	0.000	201.246
1101 → 1200	192.847	0.000	0.000	0.000	192.847
1201 → 1300	166.058	0.000	0.000	0.000	166.058
1301 → 1400	175.814	0.000	0.000	0.000	175.814
1401 → 1500	166.327	0.000	0.000	0.000	166.327
1501 → 1600	180.553	0.000	0.000	0.000	180.553
1601 → 1700	171.431	0.000	0.000	0.000	171.431
1701 → 1800	179.529	0.000	0.000	0.000	179.529
1801 → 1900	187.756	0.000	0.000	0.000	187.756
1901 → 2000	151.194	0.000	0.000	0.000	151.194
Total	3488.226	14.850	3.202	0.092	3506.186
Average Annual Cost	1.744	0.007	0.002	0.000	1.753

Table 5.61: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 12500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	153.845	0.000	0.000	0.000	153.845
101 → 200	162.125	8.380	0.000	0.000	170.505
201 → 300	191.745	0.000	0.000	0.000	191.745
301 → 400	165.936	6.470	0.000	0.000	172.406
401 → 500	181.904	0.000	0.000	0.000	181.904
501 → 600	154.246	0.000	0.000	0.000	154.246
601 → 700	184.290	0.000	7.184	0.202	191.272
701 → 800	181.745	0.000	0.000	0.000	181.745
801 → 900	161.613	0.000	0.000	0.000	161.613
901 → 1000	168.876	0.000	0.000	0.000	168.876
1001 → 1100	200.035	0.000	0.000	0.000	200.035
1101 → 1200	191.765	0.000	3.744	0.104	195.405
1201 → 1300	165.114	0.000	0.000	0.000	165.114
1301 → 1400	175.128	0.000	0.000	0.000	175.128
1401 → 1500	165.332	0.000	0.276	0.012	165.596
1501 → 1600	179.535	0.000	0.000	0.000	179.535
1601 → 1700	170.543	0.000	0.166	0.008	170.701
1701 → 1800	180.562	0.000	0.000	0.000	180.562
1801 → 1900	186.942	0.000	0.000	0.000	186.942
1901 → 2000	155.358	0.000	0.000	0.000	155.358
Total	3476.639	14.850	11.370	0.326	3502.533
Average Annual Cost	1.738	0.007	0.006	0.000	1.751

Table 5.62: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 11500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	153.065	0.000	0.000	0.000	153.065
101 → 200	161.375	8.380	0.321	0.012	170.064
201 → 300	190.723	0.000	0.000	0.000	190.723
301 → 400	165.070	6.470	0.000	0.000	171.540
401 → 500	181.085	0.000	0.000	0.000	181.085
501 → 600	153.436	0.000	0.279	0.011	153.704
601 → 700	183.181	0.000	8.979	0.272	191.888
701 → 800	180.966	0.000	0.435	0.017	181.384
801 → 900	160.840	0.000	0.000	0.000	160.840
901 → 1000	168.144	0.000	0.994	0.040	169.098
1001 → 1100	198.838	0.000	0.484	0.020	199.302
1101 → 1200	190.542	0.000	15.943	0.364	206.121
1201 → 1300	164.246	0.000	0.000	0.000	164.246
1301 → 1400	174.457	0.000	0.998	0.040	175.415
1401 → 1500	164.346	0.000	3.784	0.088	168.042
1501 → 1600	178.609	0.000	1.618	0.064	180.163
1601 → 1700	169.691	0.000	1.408	0.048	171.051
1701 → 1800	179.561	0.000	0.000	0.000	179.561
1801 → 1900	186.122	0.000	0.205	0.008	186.319
1901 → 2000	154.592	0.000	0.000	0.000	154.592
Total	3458.889	14.850	35.448	0.984	3508.203
Average Annual Cost	1.729	0.007	0.018	0.001	1.754

Table 5.63: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 10500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	152.417	0.000	0.000	0.000	152.417
101 → 200	160.538	8.380	2.208	0.072	171.054
201 → 300	189.778	0.000	1.022	0.032	190.768
301 → 400	164.041	6.470	3.714	0.112	174.113
401 → 500	180.270	0.000	0.478	0.020	180.728
501 → 600	152.825	0.000	0.963	0.028	153.760
601 → 700	181.966	0.000	15.403	0.456	196.913
701 → 800	180.202	0.000	2.233	0.072	182.363
801 → 900	160.000	0.000	0.000	0.000	160.000
901 → 1000	167.494	0.000	1.762	0.056	169.200
1001 → 1100	197.603	0.000	8.116	0.244	205.475
1101 → 1200	189.044	0.000	27.984	0.832	216.196
1201 → 1300	163.515	0.000	0.166	0.008	163.673
1301 → 1400	173.852	0.000	3.345	0.096	177.101
1401 → 1500	163.437	0.000	8.101	0.212	171.326
1501 → 1600	177.498	0.000	8.956	0.288	186.166
1601 → 1700	168.719	0.000	9.142	0.284	177.577
1701 → 1800	178.333	0.000	6.588	0.212	184.709
1801 → 1900	186.942	0.000	3.764	0.014	190.692
1901 → 2000	153.882	0.000	0.000	0.000	153.882
Total	3442.356	14.850	103.945	3.038	3558.113
Average Annual Cost	1.721	0.007	0.052	0.002	1.779

Table 5.64: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 9500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	151.851	0.000	1.052	0.040	152.863
101 → 200	159.588	8.380	10.777	0.304	178.441
201 → 300	188.626	0.000	10.227	0.336	198.517
301 → 400	162.878	6.470	19.000	0.440	187.908
401 → 500	179.351	0.000	8.830	0.276	187.905
501 → 600	152.205	0.000	6.680	0.220	158.665
601 → 700	180.734	0.000	25.343	0.772	205.305
701 → 800	179.057	0.000	13.587	0.420	192.224
801 → 900	159.191	0.000	2.502	0.096	161.597
901 → 1000	166.938	0.000	3.623	0.120	170.441
1001 → 1100	196.270	0.000	26.603	0.768	222.105
1101 → 1200	186.796	0.000	61.319	1.836	246.279
1201 → 1300	162.853	0.000	3.991	0.120	166.724
1301 → 1400	173.109	0.000	13.386	0.408	186.087
1401 → 1500	162.478	0.000	17.154	0.432	179.200
1501 → 1600	175.941	0.000	31.230	0.892	206.279
1601 → 1700	167.437	0.000	34.031	0.996	200.472
1701 → 1800	176.650	0.000	34.882	1.040	210.492
1801 → 1900	183.896	0.000	20.337	0.648	203.585
1901 → 2000	153.220	0.000	3.269	0.128	156.361
Total	3419.069	14.850	347.823	10.292	3771.450
Average Annual Cost	1.710	0.007	0.174	0.005	1.886

Table 5.65: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 8500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	151.144	0.000	10.386	0.316	161.214
101 → 200	158.716	8.380	20.771	0.620	187.247
201 → 300	186.987	0.000	34.532	1.040	220.479
301 → 400	161.914	6.470	26.108	0.760	193.732
401 → 500	178.057	0.000	31.493	0.972	208.578
501 → 600	151.467	0.000	18.093	0.556	169.004
601 → 700	178.871	0.000	59.582	1.776	236.677
701 → 800	176.827	0.000	50.533	1.584	225.776
801 → 900	158.381	0.000	9.899	0.316	167.964
901 → 1000	165.987	0.000	15.852	0.508	181.331
1001 → 1100	194.949	0.000	45.331	1.308	238.972
1101 → 1200	182.870	0.000	139.384	4.036	318.218
1201 → 1300	161.974	0.000	20.576	0.608	181.942
1301 → 1400	172.022	0.000	27.602	0.908	198.716
1401 → 1500	161.283	0.000	35.815	0.948	196.150
1501 → 1600	173.875	0.000	84.651	2.032	256.494
1601 → 1700	165.377	0.000	94.648	2.352	257.673
1701 → 1800	174.477	0.000	79.514	2.300	251.691
1801 → 1900	181.926	0.000	58.188	1.732	238.382
1901 → 2000	152.418	0.000	13.568	0.452	165.534
Total	3389.522	14.850	876.526	25.124	4255.774
Average Annual Cost	1.695	0.007	0.438	0.013	2.128

Table 5.66: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 7500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	148.492	0.000	25.165	0.736	172.921
101 → 200	157.785	8.380	35.174	1.056	200.283
201 → 300	184.640	0.000	83.372	2.488	265.524
301 → 400	160.604	6.470	55.028	1.524	220.578
401 → 500	176.369	0.000	69.296	2.072	243.593
501 → 600	150.558	0.000	37.741	1.128	187.171
601 → 700	175.066	0.000	177.489	4.104	348.451
701 → 800	174.084	0.000	111.228	3.224	282.088
801 → 900	157.570	0.000	23.123	0.764	179.929
901 → 1000	164.573	0.000	43.475	1.292	206.756
1001 → 1100	192.743	0.000	99.805	2.588	289.960
1101 → 1200	179.103	0.000	250.217	6.444	422.876
1201 → 1300	160.717	0.000	49.889	1.452	209.154
1301 → 1400	170.633	0.000	75.518	1.816	244.335
1401 → 1500	160.334	0.000	51.899	1.468	210.765
1501 → 1600	171.796	0.000	133.522	3.172	302.146
1601 → 1700	162.840	0.000	182.759	3.992	341.607
1701 → 1800	171.650	0.000	140.126	4.140	307.636
1801 → 1900	179.467	0.000	110.220	3.164	286.523
1901 → 2000	151.257	0.000	43.944	1.292	193.909
Total	3350.281	14.850	1798.990	47.916	5116.205
Average Annual Cost	1.675	0.007	0.899	0.024	2.558

Table 5.67: Restriction Policy - Case 1 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 6500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	0	0	0	0
101 → 200	0	0	0	0
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	0	0	0	0
601 → 700	0	2	0	1
701 → 800	0	0	0	0
801 → 900	0	0	0	0
901 → 1000	0	0	0	0
1001 → 1100	0	0	0	0
1101 → 1200	0	0	0	0
1201 → 1300	0	0	0	0
1301 → 1400	0	0	0	0
1401 → 1500	0	0	0	0
1501 → 1600	0	0	0	0
1601 → 1700	0	0	0	0
1701 → 1800	0	0	0	0
1801 → 1900	0	0	0	0
1901 → 2000	0	0	0	0
Total	0	2	0	1
Average Annual Occurrence	0.000	0.001	0.000	0.001
Lowest Myponga Reservoir Level attained (ML)	5298			

Table 5.68: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 12500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	0	0	0	0
101 → 200	0	0	0	0
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	0	0	0	0
601 → 700	0	3	0	1
701 → 800	0	0	0	0
801 → 900	0	0	0	0
901 → 1000	0	0	0	0
1001 → 1100	0	0	0	0
1101 → 1200	2	2	0	1
1201 → 1300	0	0	0	0
1301 → 1400	0	0	0	0
1401 → 1500	3	0	0	1
1501 → 1600	0	0	0	0
1601 → 1700	1	0	0	1
1701 → 1800	0	0	0	0
1801 → 1900	0	0	0	0
1901 → 2000	0	0	0	0
Total	6	5	0	4
Average Annual Occurrence	0.003	0.003	0.000	0.002
Lowest Myponga Reservoir Level attained (ML)	5370			

Table 5.69: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 11500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	0	0	0	0
101 → 200	2	0	0	1
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	2	0	0	1
601 → 700	3	3	0	2
701 → 800	2	0	0	1
801 → 900	0	0	0	0
901 → 1000	3	0	0	1
1001 → 1100	3	0	0	2
1101 → 1200	15	3	2	7
1201 → 1300	0	0	0	0
1301 → 1400	3	0	0	1
1401 → 1500	3	1	2	2
1501 → 1600	6	0	0	3
1601 → 1700	2	1	0	1
1701 → 1800	0	0	0	0
1801 → 1900	2	0	0	2
1901 → 2000	0	0	0	0
Total	46	8	4	24
Average Annual Occurrence	0.023	0.004	0.002	0.012
Lowest Myponga Reservoir Level attained (ML)	4982			

Table 5.70: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 10500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	0	0	0	0
101 → 200	3	2	0	2
201 → 300	4	1	0	3
301 → 400	2	4	0	3
401 → 500	3	0	0	2
501 → 600	0	2	0	1
601 → 700	6	6	0	4
701 → 800	1	2	0	1
801 → 900	0	0	0	0
901 → 1000	1	2	0	1
1001 → 1100	2	8	0	4
1101 → 1200	13	23	0	10
1201 → 1300	1	0	0	1
1301 → 1400	1	2	0	1
1401 → 1500	1	4	2	2
1501 → 1600	6	6	0	4
1601 → 1700	8	2	1	5
1701 → 1800	10	3	0	5
1801 → 1900	9	1	0	4
1901 → 2000	0	0	0	0
Total	71	68	3	53
Average Annual Occurrence	0.036	0.034	0.002	0.027
Lowest Myponga Reservoir Level attained (ML)	4789			

Table 5.71: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 9500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	4	0	0	2
101 → 200	1	7	0	3
201 → 300	11	6	0	7
301 → 400	6	5	3	5
401 → 500	9	8	0	8
501 → 600	6	3	0	3
601 → 700	23	9	1	11
701 → 800	13	7	0	7
801 → 900	9	0	0	3
901 → 1000	3	4	0	3
1001 → 1100	15	8	3	9
1101 → 1200	27	33	2	16
1201 → 1300	2	2	0	2
1301 → 1400	5	6	0	3
1401 → 1500	7	4	4	5
1501 → 1600	12	9	2	7
1601 → 1700	8	13	1	6
1701 → 1800	23	15	2	13
1801 → 1900	17	11	0	9
1901 → 2000	7	0	0	3
Total	208	150	18	125
Average Annual Occurrence	0.104	0.075	0.009	0.063
Lowest Myponga Reservoir Level attained (ML)	4816			

Table 5.72: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 8500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	8	5	1	5
101 → 200	4	11	0	4
201 → 300	22	15	2	13
301 → 400	7	10	1	5
401 → 500	16	13	1	9
501 → 600	6	8	0	4
601 → 700	19	28	2	14
701 → 800	30	21	0	14
801 → 900	9	8	0	6
901 → 1000	10	8	0	6
1001 → 1100	12	17	3	10
1101 → 1200	36	48	6	19
1201 → 1300	14	6	1	8
1301 → 1400	21	8	0	6
1401 → 1500	9	12	2	6
1501 → 1600	19	17	4	10
1601 → 1700	16	18	4	10
1701 → 1800	35	24	5	16
1801 → 1900	25	24	1	15
1901 → 2000	7	6	0	4
Total	325	307	33	184
Average Annual Occurrence	0.163	0.154	0.017	0.092
Lowest Myponga Reservoir Level attained (ML)	4625			

Table 5.73: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 7500 ML.

Water Year Range	Months at Restriction Level			Restriction Events	Myponga Failure Events
	1	2	3		
1 → 100	8	15	0	7	0
101 → 200	6	14	0	4	0
201 → 300	30	36	0	18	0
301 → 400	11	19	2	9	0
401 → 500	28	16	4	12	0
501 → 600	9	12	0	6	0
601 → 700	21	43	13	14	0
701 → 800	27	43	2	17	0
801 → 900	14	12	0	7	0
901 → 1000	13	19	0	9	0
1001 → 1100	13	27	6	12	0
1101 → 1200	34	67	12	22	1
1201 → 1300	10	22	0	9	0
1301 → 1400	17	15	4	6	0
1401 → 1500	7	21	2	8	0
1501 → 1600	17	31	5	12	0
1601 → 1700	23	23	9	11	3
1701 → 1800	36	54	0	17	0
1801 → 1900	26	41	2	20	0
1901 → 2000	19	16	0	10	0
Total	369	546	61	230	4
Average Annual Occurrence	0.185	0.273	0.031	0.115	0.002
Lowest Myponga Reservoir Level attained (ML)	4465				

Table 5.74: Restriction Policy - Case 1 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 6500 ML.

The water supply costs for the range of operating rules considered for the southern system are summarised in Table 5.75. The imposed restriction levels associated with these operating rules are summarised in Table 5.76.

Myponga Nominal Winter Minimum Operating Level (ML)	Average Annual Pumping Costs ($M)	Average Annual On-line Tankering Costs ($M)	Average Annual Economic Restriction Costs ($M)	Average Annual Reduction in Water Filtration Plant Costs ($M)	Average Annual Total Water Supply Costs ($M)
12500	1.744	0.007	0.002	0.000	1.753
11500	1.738	0.007	0.006	0.000	1.751
10500	1.729	0.007	0.018	0.001	1.754
9500	1.721	0.007	0.052	0.002	1.779
8500	1.710	0.007	0.174	0.005	1.886
7500	1.695	0.007	0.438	0.013	2.128
6500	1.675	0.007	0.899	0.024	2.558

Table 5.75: Restriction Policy - Case 1 - Average Annual Cost Summary

Myponga Nominal Winter Minimum Operating Level (ML)	Average Annual Months at Restriction Level			Average Annual Number of Restriction Events	Average Annual Number of Myponga Failure Events
	1	2	3		
12500	0.000	0.001	0.000	0.001	0.000
11500	0.003	0.003	0.000	0.002	0.000
10500	0.023	0.004	0.002	0.012	0.000
9500	0.036	0.034	0.002	0.027	0.000
8500	0.104	0.075	0.009	0.063	0.000
7500	0.163	0.154	0.017	0.091	0.000
6500	0.185	0.273	0.031	0.115	0.002

Table 5.76: Restriction Policy - Case 1 - Average Annual Restriction and Myponga Failure Summary

Examination of Tables 5.75 and 5.76 reveals a number of important results.

1. The use of a restriction policy for the southern Adelaide system can reduce system failures to zero for all operating rule sets considered except when the Myponga winter minimum is reduced to 6500 ML.

2. The economic costs associated with the imposition of water restrictions on the whole of the southern system have significant impact on the selection of an economically optimum nominal winter minimum operating level for Myponga Reservoir as part of the southern system operating rules. From the results presented in Table 5.75, a Myponga nominal winter minimum operating level of 11500 ML appears optimum over the range of operating rule sets considered.

3. The economic costs associated with the application of a restriction policy are high and with reference to the southern Adelaide system it appears appropriate to limit the application of these restriction policies to those demand areas where the application will result in a reduction in the demand on the Myponga Reservoir only.

5.4.2 Southern System Restriction Policy - Case 2

In this second case study, a range of operating rules have been considered for the southern system and a restriction policy used to reduce demand from those areas supplied solely from the Myponga Reservoir. These restrictions are imposed when the Myponga Reservoir storage level falls below the set of target storage levels. This is the preferred strategy for the imposition of restrictions, since only those demand areas reliant solely on the Myponga Reservoir for their water supply need be restricted in the event of the storage level in Myponga Reservoir falling to low levels.

The imposition and relaxation trigger storage levels for Myponga Reservoir used in this case study are the same as those used in the previous case study and are presented in Table 5.59. The reduction in outdoor water use from the Myponga and Encounter Bay demand zones associated with the three restriction classes is shown in Table 5.60 together with the economic costs per unit volume of water at each restriction level.

In this second case study, the Myponga nominal winter minimum operating level has been varied and the reliability-cost tradeoffs examined using 2000 years of synthetic inflow and demand data in conjunction with Monte Carlo failure data for the Murray Bridge-Onkaparinga pumping system. The 70% inflow exceedance sets and a '4 week demand' component of the target storages for the southern system reservoirs have been used for each operating rule sets considered. The demand forecast set adopted in this comparison is given in Table 4.11. The nominal minimum operating levels used for the Mount Bold and Happy Valley Reservoirs are those given in Table B.4. Other details used in the HOMA model are given in Appendix B.

Water supply costs for Myponga nominal winter minimum operating levels varying from 12500 ML to 6500 ML are presented in Tables 5.77, 5.78, 5.79, 5.80,5.81, 5.82 and 5.83. The number and level of restrictions imposed on the system for each of the operating rules considered are presented in Tables 5.84, 5.85, 5.86, 5.87, 5.88, 5.89 and 5.90.

With the application of the proposed water restriction policy, no failures occurred in Mount Bold, Happy Valley or Myponga Reservoirs for all operating rule sets considered except when the Myponga winter minimum was reduced to 6500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	154.733	0.000	0.000	0.000	154.733
101 → 200	162.979	8.380	0.000	0.000	171.359
201 → 300	192.746	0.000	0.000	0.000	192.746
301 → 400	166.860	6.470	0.000	0.000	173.330
401 → 500	182.830	0.000	0.000	0.000	182.830
501 → 600	154.913	0.000	0.000	0.000	154.913
601 → 700	185.709	0.000	1.018	0.028	186.699
701 → 800	182.585	0.000	0.000	0.000	182.585
801 → 900	162.523	0.000	0.000	0.000	162.523
901 → 1000	169.682	0.000	0.000	0.000	169.682
1001 → 1100	201.246	0.000	0.000	0.000	201.246
1101 → 1200	192.847	0.000	0.000	0.000	192.847
1201 → 1300	166.058	0.000	0.000	0.000	166.058
1301 → 1400	175.814	0.000	0.000	0.000	175.814
1401 → 1500	166.327	0.000	0.000	0.000	166.327
1501 → 1600	180.553	0.000	0.000	0.000	180.553
1601 → 1700	171.431	0.000	0.000	0.000	171.431
1701 → 1800	179.529	0.000	0.000	0.000	179.529
1801 → 1900	187.756	0.000	0.000	0.000	187.756
1901 → 2000	151.194	0.000	0.000	0.000	151.194
Total	3488.315	14.850	1.018	0.028	3504.155
Average Annual Cost	1.744	0.007	0.001	0.000	1.752

Table 5.77: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 12500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	153.845	0.000	0.000	0.000	153.845
101 → 200	162.125	8.380	0.000	0.000	170.515
201 → 300	191.745	0.000	0.000	0.000	191.745
301 → 400	165.936	6.470	0.000	0.000	172.406
401 → 500	181.904	0.000	0.000	0.000	181.904
501 → 600	154.246	0.000	0.000	0.000	154.246
601 → 700	184.503	0.000	2.105	0.060	186.548
701 → 800	181.745	0.000	0.000	0.000	181.745
801 → 900	161.613	0.000	0.000	0.000	161.613
901 → 1000	168.876	0.000	0.000	0.000	168.876
1001 → 1100	200.035	0.000	0.000	0.000	200.035
1101 → 1200	191.880	0.000	1.027	0.028	192.879
1201 → 1300	165.114	0.000	0.000	0.000	165.114
1301 → 1400	175.128	0.000	0.000	0.000	175.128
1401 → 1500	165.343	0.000	0.110	0.004	165.449
1501 → 1600	179.535	0.000	0.000	0.000	179.535
1601 → 1700	168.719	0.000	0.102	0.004	168.817
1701 → 1800	180.562	0.000	0.000	0.000	180.562
1801 → 1900	186.942	0.000	0.000	0.000	186.942
1901 → 2000	155.358	0.000	0.000	0.000	155.358
Total	3475.154	14.850	3.344	0.096	3493.252
Average Annual Cost	1.738	0.007	0.002	0.000	1.747

Table 5.78: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 11500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	153.065	0.000	0.000	0.000	153.065
101 → 200	161.387	8.380	0.107	0.004	169.870
201 → 300	190.723	0.000	0.000	0.000	190.723
301 → 400	165.070	6.470	0.000	0.000	171.540
401 → 500	181.085	0.000	0.000	0.000	181.085
501 → 600	153.436	0.000	0.107	0.004	153.539
601 → 700	183.477	0.000	2.525	0.076	185.926
701 → 800	180.986	0.000	0.107	0.004	181.089
801 → 900	160.840	0.000	0.000	0.000	160.840
901 → 1000	168.186	0.000	0.301	0.012	168.475
1001 → 1100	198.852	0.000	0.209	0.008	199.069
1101 → 1200	190.929	0.000	4.481	0.104	195.306
1201 → 1300	164.246	0.000	0.000	0.000	164.246
1301 → 1400	174.497	0.000	0.301	0.012	174.786
1401 → 1500	164.430	0.000	1.508	0.032	165.906
1501 → 1600	178.674	0.000	0.510	0.020	179.164
1601 → 1700	169.726	0.000	0.664	0.024	170.366
1701 → 1800	179.561	0.000	0.000	0.000	179.561
1801 → 1900	186.124	0.000	0.107	0.004	186.227
1901 → 2000	154.592	0.000	0.000	0.000	154.592
Total	3459.886	14.850	10.927	0.304	3485.359
Average Annual Cost	1.730	0.007	0.006	0.000	1.743

Table 5.79: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 10500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	152.417	0.000	0.000	0.000	152.417
101 → 200	160.609	8.380	0.772	0.024	169.737
201 → 300	189.797	0.000	0.470	0.016	190.251
301 → 400	164.148	6.470	1.271	0.036	171.853
401 → 500	180.284	0.000	0.209	0.008	180.485
501 → 600	152.848	0.000	0.370	0.012	153.206
601 → 700	182.451	0.000	4.437	0.136	186.752
701 → 800	180.276	0.000	0.617	0.020	180.873
801 → 900	160.000	0.000	0.000	0.000	160.000
901 → 1000	167.554	0.000	0.564	0.016	168.102
1001 → 1100	197.796	0.000	2.758	0.080	200.474
1101 → 1200	189.902	0.000	7.963	0.236	197.629
1201 → 1300	163.519	0.000	0.102	0.004	169.303
1301 → 1400	173.919	0.000	1.024	0.028	174.915
1401 → 1500	163.645	0.000	2.887	0.072	175.646
1501 → 1600	177.785	0.000	2.742	0.088	180.439
1601 → 1700	168.943	0.000	3.325	0.096	172.172
1701 → 1800	178.556	0.000	2.093	0.068	180.581
1801 → 1900	185.399	0.000	1.362	0.048	186.713
1901 → 2000	153.882	0.000	0.000	0.000	153.882
Total	3443.670	14.850	32.966	0.988	3490.498
Average Annual Cost	1.722	0.007	0.016	0.000	1.745

Table 5.80: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 9500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	151.877	0.000	0.403	0.016	152.264
101 → 200	159.884	8.380	3.195	0.088	171.371
201 → 300	188.862	0.000	3.491	0.112	192.241
301 → 400	163.271	6.470	6.422	0.144	176.019
401 → 500	179.589	0.000	3.006	0.092	182.503
501 → 600	152.374	0.000	2.245	0.072	154.547
601 → 700	181.542	0.000	7.395	0.220	188.717
701 → 800	179.473	0.000	4.049	0.124	183.398
801 → 900	159.281	0.000	0.712	0.028	159.965
901 → 1000	167.046	0.000	1.230	0.040	168.236
1001 → 1100	196.869	0.000	7.477	0.216	204.130
1101 → 1200	188.726	0.000	17.419	0.516	205.629
1201 → 1300	162.943	0.000	1.537	0.048	164.432
1301 → 1400	173.462	0.000	3.345	0.104	176.703
1401 → 1500	162.929	0.000	5.427	0.132	168.224
1501 → 1600	176.861	0.000	8.265	0.236	184.890
1601 → 1700	168.210	0.000	10.113	0.288	178.035
1701 → 1800	177.624	0.000	9.678	0.288	187.014
1801 → 1900	184.526	0.000	6.163	0.196	190.493
1901 → 2000	153.256	0.000	1.014	0.040	154.230
Total	3428.605	14.850	102.586	3.000	3543.041
Average Annual Cost	1.714	0.007	0.051	0.002	1.772

Table 5.81: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 8500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	151.417	0.000	3.523	0.100	154.840
101 → 200	159.316	8.380	6.387	0.373	173.710
201 → 300	187.946	0.000	11.380	0.328	198.998
301 → 400	162.625	6.470	8.151	0.232	177.014
401 → 500	178.966	0.000	8.942	0.272	187.636
501 → 600	151.926	0.000	5.195	0.160	156.961
601 → 700	180.680	0.000	16.639	0.484	196.835
701 → 800	178.535	0.000	14.064	0.440	192.159
801 → 900	158.631	0.000	2.879	0.092	161.418
901 → 1000	166.408	0.000	5.021	0.156	171.273
1001 → 1100	196.146	0.000	11.758	0.340	207.564
1101 → 1200	187.350	0.000	35.770	1.028	222.092
1201 → 1300	162.389	0.000	6.617	0.188	168.818
1301 → 1400	172.940	0.000	7.707	0.252	180.395
1401 → 1500	162.230	0.000	10.665	0.280	172.615
1501 → 1600	175.924	0.000	21.165	0.512	196.577
1601 → 1700	167.408	0.000	26.487	0.640	193.255
1701 → 1800	176.735	0.000	21.387	0.612	197.510
1801 → 1900	183.487	0.000	16.747	0.492	199.742
1901 → 2000	152.634	0.000	4.023	0.132	156.525
Total	3413.693	14.850	244.507	7.113	3665.937
Average Annual Cost	1.707	0.007	0.122	0.004	1.833

Table 5.82: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 7500 ML.

Water Year Range	Pumping Electricity Costs ($M)	Online Supply Tankering Costs ($M)	Economic Restriction Costs ($M)	Reduction in Water Filtration Costs ($M)	Total Water Supply Costs excluding Filtration ($M)
1 → 100	151.101	0.000	7.429	0.216	158.314
101 → 200	158.880	8.380	9.723	0.288	176.695
201 → 300	187.037	0.000	24.406	0.724	210.719
301 → 400	162.026	6.470	16.235	0.440	184.291
401 → 500	178.256	0.000	18.637	0.552	196.341
501 → 600	151.505	0.000	9.782	0.292	160.995
601 → 700	179.352	0.000	45.471	1.060	223.763
701 → 800	177.573	0.000	29.544	0.856	206.261
801 → 900	158.117	0.000	6.631	0.216	164.532
901 → 1000	165.761	0.000	12.131	0.360	177.532
1001 → 1100	195.272	0.000	26.321	0.680	220.913
1101 → 1200	185.998	0.000	64.498	1.640	248.856
1201 → 1300	161.779	0.000	15.100	0.436	176.443
1301 → 1400	172.413	0.000	20.575	0.492	192.496
1401 → 1500	161.628	0.000	15.779	0.444	176.963
1501 → 1600	175.133	0.000	33.089	0.800	207.422
1601 → 1700	166.481	0.000	48.294	1.076	213.699
1701 → 1800	175.631	0.000	38.309	1.128	212.812
1801 → 1900	182.519	0.000	30.358	0.872	212.005
1901 → 2000	152.025	0.000	12.161	0.356	163.830
Total	3398.487	14.850	484.473	12.928	3884.882
Average Annual Cost	1.699	0.007	0.242	0.006	1.942

Table 5.83: Restriction Policy - Case 2 - Water Supply Costs Myponga Nominal Winter Minimum Operating Level = 6500 ML.

Water Year Range	Months at Restriction Level			Restriction Events
	1	2	3	
1 → 100	0	0	0	0
101 → 200	0	0	0	0
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	0	0	0	0
601 → 700	0	2	0	1
701 → 800	0	0	0	0
801 → 900	0	0	0	0
901 → 1000	0	0	0	0
1001 → 1100	0	0	0	0
1101 → 1200	0	0	0	0
1201 → 1300	0	0	0	0
1301 → 1400	0	0	0	0
1401 → 1500	0	0	0	0
1501 → 1600	0	0	0	0
1601 → 1700	0	0	0	0
1701 → 1800	0	0	0	0
1801 → 1900	0	0	0	0
1901 → 2000	0	0	0	0
Total	0	2	0	1
Average Annual Occurrence	0.000	0.001	0.000	0.001
Lowest Myponga Reservoir Level attained (ML)	5298			

Table 5.84: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 12500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	0	0	0	0
101 → 200	0	0	0	0
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	0	0	0	0
601 → 700	0	3	0	1
701 → 800	0	0	0	0
801 → 900	0	0	0	0
901 → 1000	0	0	0	0
1001 → 1100	0	0	0	0
1101 → 1200	2	2	0	1
1201 → 1300	0	0	0	0
1301 → 1400	0	0	0	0
1401 → 1500	3	0	0	1
1501 → 1600	0	0	0	0
1601 → 1700	1	0	0	1
1701 → 1800	0	0	0	0
1801 → 1900	0	0	0	0
1901 → 2000	0	0	0	0
Total	6	5	0	4
Average Annual Occurrence	0.003	0.003	0.000	0.002
Lowest Myponga Reservoir Level attained (ML)	5370			

Table 5.85: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 11500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	0	0	0	0
101 → 200	2	0	0	1
201 → 300	0	0	0	0
301 → 400	0	0	0	0
401 → 500	0	0	0	0
501 → 600	2	0	0	1
601 → 700	3	3	0	2
701 → 800	2	0	0	1
801 → 900	0	0	0	0
901 → 1000	3	0	0	1
1001 → 1100	3	0	0	2
1101 → 1200	15	3	2	7
1201 → 1300	0	0	0	0
1301 → 1400	3	0	0	1
1401 → 1500	3	1	2	2
1501 → 1600	6	0	0	3
1601 → 1700	2	1	0	1
1701 → 1800	0	0	0	0
1801 → 1900	2	0	0	2
1901 → 2000	0	0	0	0
Total	46	8	4	24
Average Annual Occurrence	0.023	0.004	0.002	0.012
Lowest Myponga Reservoir Level attained (ML)	4982			

Table 5.86: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 10500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	0	0	0	0
101 → 200	3	2	0	2
201 → 300	4	1	0	3
301 → 400	2	4	0	3
401 → 500	3	0	0	2
501 → 600	0	2	0	1
601 → 700	6	6	0	4
701 → 800	1	2	0	1
801 → 900	0	0	0	0
901 → 1000	1	2	0	1
1001 → 1100	2	8	0	4
1101 → 1200	13	23	0	10
1201 → 1300	1	0	0	1
1301 → 1400	1	2	0	1
1401 → 1500	1	4	2	2
1501 → 1600	6	6	0	4
1601 → 1700	8	2	1	5
1701 → 1800	10	3	0	5
1801 → 1900	9	1	0	4
1901 → 2000	0	0	0	0
Total	71	68	3	53
Average Annual Occurrence	0.036	0.034	0.002	0.027
Lowest Myponga Reservoir Level attained (ML)	4789			

Table 5.87: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 9500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	4	0	0	2
101 → 200	1	7	0	3
201 → 300	11	6	0	7
301 → 400	6	5	3	5
401 → 500	9	8	0	8
501 → 600	6	3	0	3
601 → 700	23	9	1	11
701 → 800	13	7	0	7
801 → 900	9	0	0	3
901 → 1000	3	4	0	3
1001 → 1100	15	8	3	9
1101 → 1200	27	33	2	16
1201 → 1300	2	2	0	2
1301 → 1400	5	6	0	3
1401 → 1500	7	4	4	5
1501 → 1600	12	9	2	7
1601 → 1700	8	13	1	6
1701 → 1800	23	15	2	13
1801 → 1900	17	11	0	9
1901 → 2000	7	0	0	3
Total	208	150	18	125
Average Annual Occurrence	0.104	0.075	0.009	0.063
Lowest Myponga Reservoir Level attained (ML)	4816			

Table 5.88: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 8500 ML.

Water Year Range	Months at Restriction Level 1	2	3	Restriction Events
1 → 100	8	5	1	5
101 → 200	4	11	0	4
201 → 300	22	15	2	13
301 → 400	7	10	1	5
401 → 500	16	13	1	9
501 → 600	6	8	0	4
601 → 700	19	28	2	14
701 → 800	30	21	0	14
801 → 900	9	8	0	6
901 → 1000	10	8	0	6
1001 → 1100	12	17	3	10
1101 → 1200	36	48	6	19
1201 → 1300	14	6	1	8
1301 → 1400	21	8	0	6
1401 → 1500	9	12	2	6
1501 → 1600	19	17	4	10
1601 → 1700	16	18	4	10
1701 → 1800	35	24	5	16
1801 → 1900	25	24	1	15
1901 → 2000	7	6	0	4
Total	325	307	33	184
Average Annual Occurrence	0.163	0.154	0.017	0.092
Lowest Myponga Reservoir Level attained (ML.)	4625			

Table 5.89: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 7500 ML.

Water Year Range	Months at Restriction Level			Restriction Events	Myponga Failure Events
	1	2	3		
1 → 100	8	15	0	7	0
101 → 200	6	14	0	4	0
201 → 300	30	36	0	18	0
301 → 400	11	19	2	9	0
401 → 500	28	16	4	12	0
501 → 600	9	12	0	6	0
601 → 700	21	43	13	14	0
701 → 800	27	43	2	17	0
801 → 900	14	12	0	7	0
901 → 1000	13	19	0	9	0
1001 → 1100	13	27	6	12	0
1101 → 1200	34	67	12	22	1
1201 → 1300	10	22	0	9	0
1301 → 1400	17	15	4	6	0
1401 → 1500	7	21	2	8	0
1501 → 1600	17	31	5	12	0
1601 → 1700	23	23	9	11	3
1701 → 1800	36	54	0	17	0
1801 → 1900	26	41	2	20	0
1901 → 2000	19	16	0	10	0
Total	369	546	61	230	4
Average Annual Occurrence	0.185	0.273	0.031	0.115	0.002
Lowest Myponga Reservoir Level attained (ML.)	4465				

Table 5.90: Restriction Policy - Case 2 - Imposed Restrictions Myponga Nominal Winter Minimum Operating Level = 6500 ML.

The water supply costs for the range of operating rules considered for the southern system have been summarised in Table 5.91. The imposed restriction levels associated with these operating rules are summarised in Table 5.92.

Myponga Nominal Winter Minimum Operating Level (ML)	Average Annual Pumping Costs ($M)	Average Annual On-line Tankering Costs ($M)	Average Annual Economic Restriction Costs ($M)	Average Annual Reduction in Water Filtration Plant Costs ($M)	Average Annual Total Water Supply Costs ($M)
12500	1.744	0.007	0.000	0.000	1.752
11500	1.738	0.007	0.002	0.000	1.747
10500	1.730	0.007	0.006	0.000	1.743
9500	1.722	0.007	0.016	0.001	1.745
8500	1.714	0.007	0.051	0.002	1.772
7500	1.707	0.007	0.122	0.004	1.833
6500	1.699	0.007	0.242	0.007	1.942

Table 5.91: Restriction Policy - Case 2 - Average Annual Cost Summary

Myponga Nominal Winter Minimum Operating Level (ML)	Average Annual Months at Restriction Level			Average Annual Number of Restriction Events	Average Annual Number of Myponga Failure Events
	1	2	3		
12500	0.000	0.001	0.000	0.001	0.000
11500	0.003	0.003	0.000	0.002	0.000
10500	0.023	0.004	0.002	0.012	0.000
9500	0.036	0.034	0.002	0.027	0.000
8500	0.104	0.075	0.009	0.063	0.000
7500	0.163	0.154	0.017	0.091	0.000
6500	0.185	0.273	0.031	0.115	0.002

Table 5.92: Restriction Policy - Case 2 - Average Annual Restriction and Myponga Failure Summary

Consideration of Tables 5.91 and 5.92 reveals a number of important results.

1. The use of a restriction policy for the southern Adelaide system can reduce system failures to zero for all operating rule sets considered except when the Myponga winter minimum is reduced to 6500 ML.

2. The use of the proposed water restriction policy is a costly exercise in terms of economic losses to both consumers and producers and where possible, it appears appropriate to set operating rules so as to minimise the use of these policies. In the Adelaide system, the selection of operating rules can be made so as to limit the need for the application of such policies because of the pumping capacity available from the River Murray. As the demands on the system increase into the future, the timing for augmentation of the pumping systems will need to be considered with reference to the economic costs associated with the use of a water restrictions policy.

3. When the restriction policy involves the imposition of restrictions on only those areas supplied solely by Myponga Reservoir, the economic costs of restrictions are reduced from the broader based application of restrictions to the whole of the southern system.

4. From the results presented in Table 5.91, a Myponga nominal winter minimum operating level in the range 12500 ML to 9500 ML appears appropriate with the economic optimum being 10500 ML.

5. If the operators of the southern system consider it unacceptable to draw down Myponga Reservoir below the physical minimum operating level, it is recommended that the nominal winter minimum operating level for Myponga Reservoir be set at 10500 ML and '4 weeks of demand' used as part of the target storage levels. Using the restriction policy adopted as described earlier in this section and applied only to the demands met solely from Myponga Reservoir, the frequency of failure for the southern system can be reduced to less than 1 in 2000 years (99.95%).

5.4.3 Summary

The results presented for the southern component of the Adelaide water supply system with the inclusion of a water restriction policy highlight the high economic costs associated with the imposition of restrictions on a water supply system. If these economic costs are included in the reliability-cost tradeoffs for the operation of the southern system, the nominal winter minimum operating level for Myponga Reservoir should be selected so as to minimise the need to impose water restrictions. If the adopted restriction policy involves the application of restrictions on the whole of the southern system demand zone, it is recommended that a Myponga nominal winter minimum operating level of 11500 ML should be applied. If the adopted restriction policy involves the application of restrictions on only that portion of the southern system demand met from Myponga Reservoir, it is recommended that a Myponga nominal winter minimum operating level of 10500 ML should be applied.

The use of a restriction policy is a costly exercise for both consumers and the operators of a water supply system. In many water supply systems, water restrictions are a necessity rather than an option. For the Adelaide water supply system, the need for the imposition of water restrictions can be minimised through the selection of appropriate operating rule sets. For a given level of reliability, results presented in this section highlight that when selecting an operating rule set in conjunction with the application of a restriction policy, the reductions in operating costs obtained from the selection of a less conservative operating rule set are quickly offset by the increases in costs resulting from the imposition of restrictions.

5.5 Summary of Results for the Adelaide Water Supply Headworks System

Details of the Adelaide Water Supply Headworks System have been presented in Chapter 4 of this thesis. The methodology presented in Chapter 3 has been applied to the southern and northern components of the Adelaide water supply headworks system to examine reliability-cost tradeoffs associated with the operation of these systems.

To demonstrate the impact of the three major influences on reliability-cost tradeoffs, the three components of the methodology have been applied progressively.

Firstly, a hydrologic reliability assessment has been undertaken considering only the demand and inflow variability, assuming 'perfect' performance of bulk water transfer system with no restriction policy.

Secondly, a hydrologic and bulk water transfer system reliability assessment has been undertaken, considering the demand and inflow variability and the reliability of the bulk water transfer system. In this consideration, no restriction policy has been applied.

Thirdly, all three components of the methodology have been applied.

In the following sections, conclusions drawn from the progressive application of the methodology to the southern and northern components of the Adelaide Water Supply Headworks System are presented.

5.5.1 Southern System - Hydrologic Reliability Assessment

The critical component of the southern system is Myponga Reservoir. Unlike all other reservoirs in both the southern and northern systems, the catchment runoff to Myponga Reservoir cannot be supplemented with water pumped from the River Murray. In the hydrologic reliability assessment of the southern system, perfect performance of the bulk water transfer system is assumed. Failures are defined as occurring when the storage level in a reservoir falls below the physical minimum operating level. Failure occurrences for the southern system have been determined using a 2000 year record of synthetic inflow and demand data.

A range of operating rule sets has been examined for the southern system and tradeoffs between reliability and operating cost for the system considered.

The forecast inflows used within the operating rules have been considered and the results from this examination for the pumping costs are shown in Tables 5.2, 5.3 and 5.4. Myponga failure occurrences associated with these operating rules are presented in Tables 5.5, 5.6 and 5.7. These results are plotted in Figure 5.1. The current operating rule set includes the use of a 70% forecast inflow exceedance data set. Adopting this forecast inflow set, it has been determined that the southern system will have a failure frequency of approximately 1 in 400 years (ie. 99.75% annual reliability), with an associated average failure duration of approximately 3 months. The average annual operating costs for the southern system using this operating rule set have been determined as $1.850M. If a 60% forecast inflow exceedance data set were used, this would involve a corresponding failure frequency of approximately 1 in 140 years (ie. 99.3% annual reliability) with an average failure duration of 2.1 months. The average annual operating costs for the southern system using these operating rules is $1.824M. If a reduction in reliability due to hydrologic uncertainty from 99.75% to 99.3% is acceptable, then an estimated reduction in average operating costs of around $26,000/annum can be achieved.

The target storage levels used as part of the operating rules for the Adelaide headworks system have been described in detail in Chapter 4 within Section 4.2.4.1. Both the nominal minimum operating level and the demand storage components of these target storage levels have been considered and reliability-cost tradeoffs determined for a range of operating rule sets.

Adopting a 70% inflow exceedance forecast set and holding all other operating rules constant, the demand storage component for each of the reservoirs in the southern system has been varied from '8 weeks demand' to '2 weeks demand'. Pumping costs obtained using these operating rule sets are presented in Table 5.8 and the associated Myponga failure occurrences are presented in Table 5.9.

Reducing the demand storage component from '8 weeks demand' to '4 weeks demand' produces no change in the frequency or duration of failures of the Myponga Reservoir, but reduces the average annual operating costs from $1.850M to $1.714M. Reducing the demand storage to '2 weeks demand' reduces the average annual operating costs to $1.650M but increases the frequency of failure from approximately 1 in 400 years (ie. 99.75% annual reliability) to 1 in 250 years (ie. 99.6% annual reliability) for the Myponga Reservoir.

Adopting a 70% inflow exceedance forecast set and a target storage level including '4 weeks demand' for each of the reservoirs in the southern system, the nominal winter minimum operating level component for the Myponga Reservoir has been varied from 11500 ML to 6500 ML. The pumping costs obtained from this analysis are presented in Tables 5.35 and 5.36 with the corresponding failure occurrences for Myponga Reservoir presented in Tables 5.37 and 5.38. These results highlight the critical element of the southern system, in terms of the hydrologic risks, as the nominal winter minimum operating level adopted within the Myponga Reservoir. Raising the nominal winter minimum operating level from 9500 ML to 11500 ML involves an increase in average annual operating costs from \$1.714M to \$1.730M, but an associated reduction in failure frequency in Myponga Reservoir from approximately 1 in 400 years (ie. 99.75% annual reliability) to 1 in 2000 years (ie. 99.95% annual reliability).

Adopting a 70% inflow exceedance forecast set, a target storage level including '4 weeks demand' for each of the reservoirs in the southern system and a nominal winter minimum operating level for Myponga Reservoir of 8500 ML , the nominal winter minimum operating level component for the Onkaparinga system has been varied from 4900 ML to -100 ML. (A -100 ML nominal winter minimum has been obtained by setting the Happy Valley nominal winter minimum to 0 ML and reducing the Mount Bold nominal winter minimum to -100 ML. The additional 100 ML has been obtained from the '4 week demand' component of the target storage level.). The pumping costs obtained from this analysis are presented in Tables 5.14 and 5.15 with the corresponding failure occurrences for Happy Valley Reservoir presented in Table 5.16. Reducing the nominal winter minimum operating level from 4900 ML to 900 ML produces no change in the frequency or duration of failures for the southern system. These changes result in a reduction in average annual operating costs from \$1.707M to \$1.629M. Reducing the nominal winter minimum operating level to -100 ML reduces the average annual operating costs further to \$1.608M but dramatically increases the frequency of failure in the Happy Valley and Mount Bold Reservoirs.

When assessing the impact of hydrologic variability, it is important to recognise that the parameters used in the synthetic inflow and demand data generation models will have an associated level of uncertainty. Although a rigorous analysis of these parameter uncertainties has not been undertaken, sensitivity analysis on six parameters identified as key to the reliability-cost tradeoffs for the southern system have been considered. The results presented in Section 5.2.3 highlight that uncertainty associated with the inflow parameters will have some impact on the assessed reliability-cost tradeoffs for the southern system and must be considered in the selection of the level of system reliability required by the system operators.

5.5.2 Northern System - Hydrologic Reliability Assessment

A range of operating rule sets has been examined to determine reliability-cost tradeoffs for the northern system.

The forecast inflow data sets used within the operating rules have been considered and the results from this examination for the pumping costs are shown in Tables 5.18 and 5.19. These results are plotted in Figure 5.5. There were no failure occurrences for any of the reservoirs in the northern system for each of the inflow exceedance forecast data sets examined. These results suggest that the 50% exceedance inflow forecast set could be selected for the northern system with minimal reduction in system reliability. The current operating procedure involves the use a 70% forecast inflow exceedance data set with a corresponding average annual operating cost for the northern system of $5.055M. If a 50% forecast inflow exceedance data set were used, this would involve a corresponding average annual operating cost for the northern system of $4.908M.

Both the nominal minimum operating level and the demand storage components of the target storage levels have been considered and reliability-cost tradeoffs for these components determined for the northern system.

While maintaining the nominal minimum operating level component of the target storages for each of the reservoirs in the northern system constant, the demand storage for each of the reservoirs has been varied from '8 weeks demand' to '2 weeks demand'. There were no failure occurrences for any of the reservoirs in the northern system for each of the demand storage levels examined. Reducing the demand storage from '8 weeks demand' to '2 weeks demand' produces a reduction in average annual operating costs from $5.055M to $4.721M.

Adopting a demand storage level of 4 weeks for each of the reservoirs in the northern system, the nominal winter minimum operating level component for South Para Reservoir has been varied over the range 11100 ML to 2100 ML. The pumping costs obtained from this analysis are presented in Tables 5.21 and 5.22. These results are plotted in Figure 5.6. There were no failure occurrences for any of the reservoirs in the northern system for each of the nominal winter minimum operating levels examined. Reducing the nominal winter minimum operating level for South Para Reservoir to 2100 ML produces a reduction in average annual operating costs from $4.821M to $4.682M.

Adopting a demand storage level of 4 weeks for each of the reservoirs in the northern system, the nominal winter minimum operating level component for Little Para and Millbrook Reservoirs has been varied from 5000 ML to 3000 ML and 3000 ML to 1000 ML respectively. The pumping costs obtained from this analysis are presented in Table 5.23. Reducing the nominal winter minimum operating level for Little Para and Millbrook Reservoirs to 3000 ML and 2000 ML respectively produces no change in the frequency or duration of failure for the northern system but results in a reduction in average annual operating costs from $4.821M to $4.782M. Reducing the nominal winter minimum operating levels to 2000 ML and 1000 ML respectively reduces the average annual operating costs further to $4.743M, but dramatically increases the frequency of failure in Little Para Reservoir.

5.5.3 Southern System - Hydrologic and Bulk Water Transfer System Reliability Assessment

The conclusions drawn in Section 5.2 for the southern system are based solely upon the hydrologic reliability assessment and assume the Murray Bridge-Onkaparinga pumping system operates with perfect reliability.

In Section 5.3, a range of operating rule sets have been examined for the southern system with the inclusion of the impact of reliability attributes of the Murray Bridge-Onkaparinga pumping system.

In the first examination, a demand storage level comparison is undertaken using the same range of operating rule sets considered in Section 5.2. Results from this examination indicate that no additional system failures occur with the inclusion of pumping system failures over the range of operating rules considered.

While maintaining the nominal minimum operating levels component of the target storages for each of the reservoirs in the southern system constant, the demand storage component for each of the reservoirs has been varied from '8 weeks demand' to '2 weeks demand'. Pumping costs obtained using these operating rule sets are presented in Table 5.33 and the associated Myponga failure occurrences are presented in Table 5.34.

Reducing the demand storage component to '4 weeks demand' produces no change in the frequency or duration of failure of the Myponga Reservoir but results in a reduction in average annual operating costs of \$1.866M to \$1.730M. Reducing the demand storage from '8 weeks demand' to '2 weeks demand' reduces the average annual operating costs further to \$1.667M, but at the expense of increasing the frequency of failure from approximately 1 in 400 years (ie. 99.75% annual reliability) to 1 in 250 years (ie. 99.6% annual reliability) for the Myponga Reservoir.

In the second examination of the southern system, target storage levels including '4 weeks demand' for each reservoir in the system have been adopted and the Myponga nominal winter minimum operating level has been varied from 11500 ML to 6500 ML. The pumping costs obtained from this analysis are presented in Tables 5.35 and 5.36 with the corresponding failure occurrences for Myponga Reservoir presented in Tables 5.37 and 5.38.

Results from this examination again indicate that no additional system failures over with the inclusion of pumping system failures over the range of operating rule sets considered.

The reliability parameters estimated for the Murray Bridge-Onkaparinga pumping system will have an associated level of uncertainty. In order to test the sensitivity of the results for the southern system, the parameter estimates for two critical components have been modified. Using the modified failure frequency attributes for the overall pumping system, reliability-cost tradeoffs have been examined by varying the Myponga nominal winter minimum operating levels. Results from this reliability-cost analysis have been presented in Tables 5.47, 5.49, 5.48 and 5.50 and contrasted with the unmodified reliability results in Table 5.51. These results highlight the relative insensitivity of the southern system tradeoffs to uncertainties in the component reliability parameter estimates.

These results confirm previous analyses that highlighted the critical aspect of the southern system operating rule set as the Myponga nominal winter minimum operating level. With an increase in average annual operating costs from \$1.730M to \$1.746M by raising the nominal winter minimum operating level in Myponga Reservoir from 9500 ML to 11500 ML the frequency of failure in the Myponga Reservoir can be reduced from approximately 1 in 400 years (ie. 99.75% annual reliability) to 1 in 2000 years (ie. 99.95% annual reliability).

5.5.4 Northern System - Hydrologic and Bulk Water Transfer System Reliability Assessment

In section 5.3, a range of operating rule sets have been examined for the northern system with the inclusion of the impact of the reliability of the Mannum-Adelaide and Swan Reach-Stockwell pumping systems and the Millbrook pump station.

In the first examination, a demand storage level comparison has been undertaken using the same range of operating rule sets considered in section 5.2. The pumping costs obtained from this analysis are presented in Tables 5.39. Results from this examination indicate that no additional system failures occur with the inclusion of pumping system failures over the range of operating rule sets considered. Reducing the demand storage from '8 weeks demand' to '2 weeks demand' produces a reduction in average annual operating costs from $5.072M to $4.744M.

In the second examination of the northern system, target storage levels including '4 weeks demand' for each reservoir in the system have been adopted and the South Para nominal winter minimum operating level has been varied from 11100 ML to 2100 ML. The pumping costs obtained from this analysis are presented in Tables 5.40 and 5.41. There were again no failure occurrences over the range of operating rule sets considered during the period of simulation.

In the third examination of the northern system, target storage levels including '4 weeks demand' for each reservoir in the system have been adopted and the Little Para and Millbrook nominal winter minimum operating levels have been varied. The pumping costs obtained from this analysis are presented in Tables

5.40 and 5.41.

The reliability parameters estimated for the pumping systems utilised in the northern system will have an associated level of uncertainty. In order to test the sensitivity of the results for the northern system, the parameter estimates for two critical components in the Mannum-Adelaide pumping system have been modified. Using the modified failure frequency attributes for the overall Mannum-Adelaide pumping system, together with the original failure frequency attributes for the Swan Reach-Stockwell pumping system and the Millbrook pump station, reliability-cost tradeoffs have been examined by varying the Little Para and Millbrook Reservoir nominal winter minimum operating levels.

Results from this analysis have been presented in Tables 5.55, 5.56 and 5.57 and contrasted with the unmodified reliability results in Table 5.58. These results highlight a number of important considerations in the selection of operating rules for the northern system :

1. The northern system has a greater dependence on the Mannum-Adelaide pumping system than the southern system on the Murray Bridge-Onkaparinga pumping system.

2. It is important that the reliability attributes of the bulk water transfer systems be included in the reliability-cost assessment for the Adelaide water supply system.

3. The results obtained for the northern system, with the inclusion of the reliability attributes of the bulk water transfer systems, are moderately sensitive to the assessed critical component reliability attributes. It is therefore important that the uncertainties associated with the assessment of the critical pumping system component reliability attributes be considered when selecting operating rules for the northern system.

5.5.5 Southern System - Hydrologic Variability and Bulk Water Transfer System Reliability with the application of a Restriction Policy

In Section 5.4, a restriction policy has been included in the operating rules for the southern Adelaide water supply headworks system. Using two case studies, a range of operating rule sets have been considered to determine reliability-cost tradeoffs for the southern system. The impact of hydrologic variability and bulk water transfer system reliability have been included in both case studies.

In the first of these studies, restrictions have been imposed on all of the southern system demands, in the event of Myponga Reservoir falling below a specified trigger level. In the second case study, restrictions have been imposed only on those areas supplied from Myponga Reservoir.

Using the economic costs associated with the imposition of restriction as determined using the approach presented by Dandy [66], reliability-cost tradeoffs have been determined for a range of operating rule sets involving variation of the Myponga nominal winter minimum operating level.

With the application of a water restriction policy, the number of failure events can be reduced to zero for each of the operating rules sets considered, in both case studies.

In the first case study, where restrictions were imposed on <u>all</u> southern system demands, reduction in pumping electricity costs were equalled or exceeded by increases in the costs associated with the imposition of restrictions for all operating rules sets when the Myponga nominal winter minimum operating level was reduced below 12500 ML.

If the adopted policy by the operators of the system was to impose restrictions on all southern system demands, then the economically optimum Myponga nominal winter minimum operating level using a '4 week demand' component of the target storage levels would be in the range 11500 ML to 12500 ML , with associated average annual total costs of $1.754M. For Myponga nominal winter minimum operating levels below 11500 ML , the overall annual average costs associated with the operation of the souther system rise quite rapidly, with reductions in pumping costs being swamped by increases in the costs associated with the imposition of water restrictions.

In the second case study, where restrictions were imposed on only those southern system demands met from Myponga Reservoir, the optimum Myponga nominal winter minimum operating level is marginally lower than the first case considered with the optimum in the range 10500 ML to 11500 ML , having associated average annual total costs of $1.747M to $1.748M.

5.6 Summary of Chapter

In this chapter, the methodology described in Chapter 3 has been applied to the assessment of reliability-cost tradeoffs for the operation of the metropolitan Adelaide water supply system. Following the introduction to the chapter, results from the simulation of the southern and northern components of the Adelaide system using synthetic inflow and demand data have been presented. In these simulations, the bulk water transfer system has been assumed to have 100% reliability. The examination has considered a range of operating rule sets for these systems including the inflow forecast data sets, the demand storage components of the reservoir target storage levels and the nominal winter minimum operating levels for the reservoirs. The sensitivity of the results have also been considered with respect to the uncertainty in the parameters used to generated the synthetic inflow data. Using results from these simulations, reliability-cost tradeoffs have been presented taking into account hydrologic variability in the operation of the system.

A key element in certain water supply systems is the transfer of water from a remote source. Following the consideration of the hydrologic factors affecting the operation of the Adelaide water supply system, the effect of the reliability of the bulk water transfer system on the system reliability and operating costs has been considered. Using results from the 'walking party' approach for the assessment of critical components of a bulk water transfer system, described in Chapter 4, the performance of the southern and northern components of the Adelaide system have been examined. The sensitivity of the results have also been considered with respect to the uncertainty in the component reliability parameters for the critical components in the bulk water transfer systems. Using the results from these simulations, reliability-cost tradeoffs have been presented which take into account both hydrologic variability and bulk water transfer system reliability.

A feature of many water supply systems is the use of water supply restrictions during periods of water shortages, to ensure a level of supply is maintained to consumers. In the last section of this chapter, the use of a water restriction policy has been considered as part of the operating rules for the Adelaide system. Using the methodology described in Chapter 3, the economic costs associated with the imposition of water restrictions have been included in the reliability-cost tradeoff assessment for the Adelaide system. Using results from the simulation of the southern system, reliability-cost tradeoffs have been presented, taking into account hydrologic variability, the reliability of the bulk water transfer system and the economic costs associated with the use of a water restriction policy.

This chapter has presented the results of the application of the methodology described in Chapter 3 to the Adelaide system. Results obtained for the Adelaide system have highlighted potential reductions in operating costs that may be available through the consideration of reliability-cost tradeoffs for water supply headworks systems.

Chapter 6

Conclusions and Recommendations

6.1 Conclusions

In this thesis, a methodology has been presented for the assessment of reliability-cost tradeoffs in the operation of multiple reservoir headworks systems.

The methodology considers the four major influences affecting the reliability-cost tradeoffs for a multiple reservoir water supply headworks system. These influences are :

1. The inflow variability.

2. The demand variability.

3. The reliability of a bulk water transfer system.

4. The application of water restrictions.

The use of a synthetic inflow data generation model enables the examination of inflow variability on reliability-cost tradeoffs in the operation of multiple reservoir headworks systems, and forms the first component of the methodology. In the case study of the Adelaide water supply headworks system, a synthetic inflow data generation model developed by Dandy and Baker [64] has been used to generate synthetic inflow data for the metropolitan Adelaide reservoirs. This model is a first-order multisite multiperiod autoregressive data generation model of a similar type to the model developed by Young and Pisano [329]. Although the impact of the model and data parameter uncertainties is not included within this model, sensitivity analysis has been undertaken on several of the assumed model parameters identified as having greatest impact on the reliability-cost tradeoffs. The inclusion of inflow variability on the reliability-cost tradeoffs for the water supply headworks system forms the first component of the methodology.

Demand for water is affected by a range of factors. The variability of demand from a water supply headworks system will also influence the reliability-cost tradeoffs in the operation of the system. In many water supply systems, the variability in demand will be smaller than the variability in inflow and hence have less impact on these tradeoffs. In the case study of the Adelaide water supply headworks system, a simple synthetic demand data generation model has been developed that utilises the synthetic rainfall data generated by the inflow data generation model. Additional complexity has not been included within the synthetic demand data generation model as the nature of the Adelaide system is such that the impact of demand variability is small when compared with the impact of inflow variability and the reliability of the pumping system from the River Murray. The inclusion of demand variability on the reliability-cost tradeoffs for the water supply headworks system forms the second component of the methodology.

In order to assess the reliability of a bulk water transfer system, critical components within the system need to be identified. Reliability attributes then need to be determined for these components and the impact of the reliability of these individual components on the overall bulk water transfer system determined. A technique entitled the 'walking party' approach has been developed in this research to identify these critical components and obtain realistic estimates of their reliability attributes. The 'walking party' approach involves five distinct phases. These are :

1. Establishment of an assessment framework.
2. Individual interviews of experts.
3. Review of the individual interview outcomes.
4. The 'walking party' process.
5. Preparation and review of the final report.

Application of the 'walking party' approach to the bulk water transfer system forming part of the Adelaide water supply headworks system has been described in this thesis. Results obtained from this application are also detailed.

Having identified and obtained reliability attributes for the critical components of the bulk water transfer system, these individual reliability attributes need to be combined to obtain the overall reliability attributes of the bulk water transfer system. The 'frequency-duration' analysis technique was first developed within the field of power system engineering to evaluate and compute the reliability for systems comprising electric power generation, transmission and distribution. Hobbs and Beim [14] [137] adapted this technique to the analytical simulation of a water supply system. Using 'frequency-duration' analysis, the individual component reliability attributes can be combined to obtain the overall reliability properties of a bulk water transfer system.

Having obtained the overall reliability attributes of the bulk water transfer system, the impact of these attributes needs to be considered on the reliability-cost tradeoffs for the overall headworks system. Using a Monte Carlo simulation model, random failure events can be generated for the bulk water transfer system. Using these random failure events, the performance of the water supply headworks system can be simulated and the impact of the reliability of the bulk water transfer system examined. Associated with the assessment of the individual component reliability parameters will be a level of uncertainty. Although a more rigorous assessment of the impact of these parameter uncertainties would be ideal, sensitivity analysis has been undertaken on those elements identified as having the largest impact on the reliability-cost tradeoffs for the operation of the system. The inclusion of the bulk water transfer system reliability forms the third component of the methodology.

Water restrictions are a common mechanism used by water supply authorities to reduce demand on a water supply headworks system during periods of water shortage. The use of water restrictions to reduce demand will not be without cost. Using the approach proposed by Dandy [66], the economic costs associated with the imposition of water restrictions have been considered and the impact of the use of water restrictions has been included in the assessment of reliability-cost tradeoffs for a water supply headworks system. For the Adelaide water supply system, considerable outdoor water usage occurs during the summer months. In the application of the methodology to the Adelaide system, the effect of outdoor water use restrictions has been considered. The inclusion of the economic impacts of a water restriction policy form the fourth component of the methodology.

To demonstrate the application of the methodology for the assessment of reliability-cost tradeoffs for a water supply headworks system, the Adelaide system has been examined as a case study in this thesis. Results from this examination have highlighted the potential benefits that can be obtained through the consideration of reliability-cost tradeoffs for a water supply headworks system, in which a significant proportion of the water is pumped from a distant water source. The results indicate that the inclusion of the reliability of the bulk water transfer system may be important in the overall examination of reliability-cost tradeoffs. In the case study of the Adelaide Water Supply Headworks system undertaken in this thesis, the reliability of the bulk water transfer system at the current levels of demand was shown to have only limited impact on these reliability-cost tradeoffs. As demand on the system increases in the future, the impact of the reliability of the bulk water transfer system on these reliability-cost tradeoffs will increase.

6.2 Contributions made in this Thesis

The work presented in this thesis makes a number of contributions to the body of knowledge in the area of water resources planning and management. These contributions include :

1. A methodology has been developed for the assessment of reliability-cost tradeoffs for multiple reservoir water supply headworks systems that considers inflow variability, demand variability, the reliability of a bulk water transfer system and the economic costs associated with the imposition of water restrictions. To the authors knowledge, a methodology including all four of these influences has not previously been presented.

2. Within the overall methodology, an approach has been presented entitled the 'walking party' approach for the identification and assessment of reliability attributes of critical components used to transfer water in a water supply headworks system. The 'walking party' approach provides a framework that is applicable to the reliability assessment of any system of mechanical or electrical components.

3. 'Frequency-duration' analysis has found limited application in the consideration of water supply headworks systems. In this thesis, the use of 'frequency-duration' analysis has been demonstrated for the simplification of the reliability attributes of a complex system of components connected in series and parallel. Results from the application of the analysis technique have been used to consider the impact of the reliability of the complex system on the overall reliability-cost tradeoffs for a water supply headworks system.

4. The imposition of restrictions on a water supply system will involve significant economic costs. In this thesis, the approach proposed by Dandy [66] for the assessment of these economic costs has been included in the overall methodology. This is the first application of this approach to an actual water supply system.

5. The application of the methodology for the assessment of reliability-cost tradeoffs to a multiple reservoir water supply headworks system and the demonstration of the potential reductions in operating costs that can be obtained through modification to the system operating rules with little or no reduction in system reliability.

6. A demonstration of the relative contributions towards the reliability-cost tradeoffs made by hydrologic variability, pumping system failures, and the imposition of water restrictions for a water supply headworks system in which a significant proportion of the water supply is obtained from a distant water source.

6.3 Specific Recommendations for the Adelaide Water Supply Headworks System

The results obtained from the application of the methodology to the Adelaide water supply headworks system described in Chapter 5 of this thesis has indicated significant savings are available in the operation of both the northern and southern components of the system.

Recommendations that can be drawn from the examination of the Adelaide system including the impacts of hydrologic variability and bulk water transfer system reliability are :

1. The selection of more conservative operating rules for the Adelaide water supply headworks system is preferable to the use of a water restriction policy to attain an appropriate system reliability level. There is still scope however for the reduction in operating costs through modification to the operating rule sets for both the southern and northern systems.

2. Currently 70% inflow exceedance forecast sets are used for both the southern and northern components of the Adelaide headworks system. Adjustments can be made to the forecast inflows used in the preparation of the pumping program for the northern component of the Adelaide headworks system. If 50% inflow forecasts are used instead of the current 70% forecasts, a reduction in average pumping cost of approximately $147,000/annum for the northern system will be obtained with no apparent reduction in system reliability.

3. Consideration should be given to the period of demand storage required in both the southern and northern systems. If the demand storage component of the target storage levels is reduced from 4 weeks to 2 weeks, then the average annual pumping costs will be reduced by approximately

$63,000/annum for the southern system and approximately $98,000/annum for the northern system, with only minor reduction in system reliability.

4. Adjustments be made to the South Para winter nominal minimum operating level reducing this value from 11100 ML to 4100 ML. During the 1000 year synthetic data model run using a nominal winter minimum operating level of 4100 ML , the lowest combined storage level reached in the South Para system, comprising the Warren, South Para and Barossa Reservoirs was 7283 ML which is 4300 ML above the physical minimum operating level for the reservoir system. This corresponds to over 2 weeks demand storage above the system physical minimum operating level during summer. This change in operating policy will result in a reduction in pumping cost for the northern system of approximately $112,000/annum.

5. Consideration be given to the Little Para and Millbrook nominal winter minimum operating levels. Reduction in these levels from 3000 ML and 5000 ML to 2000 ML and 4000ML respectively, including the impact of the assessed reliability attributes of the northern system pumping systems, results in a reduction in pumping cost of $42,000/annum.

6. Consideration be given to the Myponga nominal winter minimum operating level. A range of operating rule sets have been considered including the use of a restriction policy. Results from these analyses present a range of possible alternatives depending on what conditions within the southern system are considered acceptable or unacceptable to the operators of the system. These conditions include :

    (a) the reliability level required for the Myponga Reservoir,

    (b) whether draw-down of Myponga Reservoir below its minimum operating level is acceptable and

(c) the minimum storage level acceptable in Myponga Reservoir.

6.3.1 Restrictions Policies

The implementation of restriction policies is a widely used technique in times of water shortage in a water supply headworks system. Results presented in this thesis have highlighted the significant economic costs associated with the implementation of such policies, in addition to any political costs.

The inclusion of the economic costs associated with the imposition of water restrictions in the operating rule set for the southern Adelaide Water Supply System has shown that it is preferable to adopt a more conservative operating rule set that involves minimal imposition of restrictions, rather than a less conservative operating rule set that involves more frequent imposition of water restrictions.

The very high reliability of the Adelaide Water Supply Headworks System resulting from having bulk water transfer systems from the River Murray provides the option of reducing the requirement to impose water restrictions. Selection of an appropriate operating rule set can be used to take into account the effects of hydrologic variability and pumping system reliability.

The results presented for the southern Adelaide Water Supply System, including the application of a water restriction policy, reveal that temporary draw down of Myponga Reservoir below the physical minimum operating level by using temporary pumps and pipework is preferable to the imposition of water restrictions on those areas solely supplied from Myponga Reservoir.

6.4 Recommendations for Further Work

There are a number of areas in which further refinement of the methodology presented in this thesis would be beneficial.

1. Consideration of the uncertainty associated with the assessed inflow parameters used in the generation of synthetic inflow data has been undertaken in this research work through the application of sensitivity analysis. A more rigorous analysis was precluded because of the large number of parameters associated with the use of a multi-site model (180 parameters together with the monthly coefficients of the $[A]$ and $[B]$ matrices). Further work considering uncertainties associated with a larger number of parameters in synthetic inflow data generation models is required.

2. The uncertainty associated with the assessment of the reliability attributes of critical components in the bulk water transfer systems has also been considered using a sensitivity analysis. A more rigorous approach examining the effect of these parameter uncertainties would be a useful future development.

3. The 'walking party' approach presented in this thesis for the identification of the critical components in a bulk water transfer and the assessment of their reliability parameters warrants further consideration in several areas :

   (a) Examination of the accuracy of the estimates obtained using this approach, by application to a system with 'known' reliability attributes.

   (b) The extension of the approach to include confidence limits estimates by the participants about their reliability parameter estimates.

   (c) Examination of benefits that can be obtained by the improvement of the reliability attributes of specific critical components, through replacement with a more reliable equivalent component or the holding of spares in stock to reduce mean repair times.

4. It has been demonstrated that the River Murray is an extremely reliable source for the Adelaide Water Supply System. For some water supply systems dependent on a distant water source, this may not be the case. The inclusion of the effect of the reliability of the distant water source warrants further consideration. If reliability attributes for this source can be accurately assessed, then it would be possible to include the 'supply source' as an additional component within the 'frequency-duration' analysis of the bulk water transfer system.

5. The reliability of water filtration plants has not been considered in the methodology presented in this thesis. It was considered that the reliability of these plants will have limited impact on the reliability-cost tradeoffs for the operation of the water supply headworks systems. Further work is needed in this area, considering the impact of differing water filtration plant unit costs on the operation of water supply headworks systems and examination of the reliability of the operation of these plant on the overall headworks system reliability.

Bibliography

[1] Abu Rizaiza, O.S. (1991), *Residential Water Usage : A Case Study of the Major Cities of the Western Region of Saudi Arabia,* Water Resources Research, Vol. 27, No. 5, May 1991, pp. 667-671.

[2] Agardy, F.J. (1975), *Contingency Planning for Water Utilities,* Journal of the American Water Works Association, Vol. 67, No. 4, April 1975, pp. 159-163.

[3] Agthe, D.E. and Billings, R.B. (1980), *Dynamic Models of Residential Water Demand,* Water Resources Research, Vol. 16, No. 3, June 1980, pp. 476-480.

[4] Agthe, D.E., Billings, R.B., Dobra, J.L. and Raiffiee, K. (1986), *A Simultaneous Equation Demand Model for Block Rates,* Water Resources Research, Vol. 22, No. 1, January 1986, pp. 1-4.

[5] AIEE Committee (1961), *Application of Probability Methods to Generating Capacity Problems,* AIEE Transactions Power Apparatus and Systems, Vol. 79, Pt. 3, February 1961, pp. 1165-1182.

[6] Armstrong, J.S., Denniston, W.B. and Gordon, M.M. (1975), *The Use of the Decomposition Principle in Making Judgements,* Organizational Behaviour and Human Performance, Vol. 14, No. 2, October 1975, pp. 257-263.

[7] Baddeley, A. (1976), *The Psychology of Memory,* Basic Books, New York, 1976.

[8] Bain, L.J. (1978), *Statistical Analysis of Reliability and Life-Testing Models : Theory and Methods,* Marcel Dekker Inc., New York, 1978.

[9] Baker, T.J. and Dandy, G.C. (1990), *Evaluation of Multisite Models for Flow Forecasting,* Australian Water Research Advisory Council Research Project P87/17, Final Report, Department of Primary Industries and Energy, Australian Government Publishing Service, Canberra, January 1990.

[10] Baker, T.J. (1991), *Evaluation of Flow Forecasting Models for Adelaide Hill's Catchments,* A Thesis for the Degree of Master of Engineering Science. Department of Civil Engineering, University of Adelaide, November 1991.

[11] Beard, L.R., Weiss, A.C., and Austin, T.A. (1972), *Alternative Approaches to Water Resource System Simulation,* Tech. Paper 32, Hydrol. Eng. Cen., U.S. Army Corps of Engineers, Davis, California, 1972.

[12] Bechard, D., Corbu, I., Gagnon, R., Nix, G.A., Stewart, K. and Trinh, M. (1982), *ORRMS - A Computer Program with Capability for River Basin Planning Studies,* presented at the 34th Annual Conference of the Canadian Water Resources Association, Banff, Alberta, June 1982.

[13] Becker, L., and Yeh, W.W-G. (1974), *Optimisation of Real Time Operation of a Multiple-Reservoir System,* Water Resources Research, Vol. 10, No. 6, December 1974, pp. 1107-1112.

[14] Beim, G.K. and Hobbs, B.F (1988), *Analytical Simulation of Water System Capacity Reliability 2. A Markov Chain Approach and Verification of the Models,* Water Resources Research, Vol. 24, No. 9, September 1988, pp. 1445-1458.

[15] Bellman, R.E. (1957), *Dynamic Programming,* Princeton University Press, Princeton, N.J., 1957.

[16] Benson, M.A. and Matalas, N.C (1967), *Synthetic Hydrology Based on Regional Statistical Parameters,* Water Resources Research, Vol. 3, No. 4, Fourth Quarter 1967, pp. 931-935.

[17] Ben-Zvi, S (1980), *Estimates of Price and Income Elasticities of Demand for Water in Residential Use in the Red River Basin,* US Corps of Engineers, Tulsa, Oklahoma, 1980.

[18] Billinton, R., and Allan, R.N. (1984), *Evaluation of Power System Reliability,* Plenum Press, New York, 1984.

[19] Billinton, R., Allan, R.N., and Salvaderi, L. (Editors) (1991), *Applied Reliability Assessment in Electric Power Systems,* The Institute of Electrical and Electronics Engineers, New York, July 1991.

[20] Biswas, A.K. (1976), *Systems Approach to Water Management,* McGraw-Hill, New York, 1976.

[21] Boland, J.J., Carver, P.H. and Flynn, C.R. (1980), *How Much Water Supply Capacity is Enough?,* Journal of the American Water Works Association, Vol. 72, No. 7, July 1980, pp. 367-374.

[22] Bower, B.T., Hufschmidt, M.M. and Reedy, W.H. (1962), *Operating Procedures : Their Role in the Design and Implementation of Water Resource Systems by Simulation Analysis,* in Design of Water Resource Systems : new techniques for relating economic objectives, engineering analysis, and governmental planning, Chapter 11, edited by A. Maass et al., Harvard University Press, Massachusetts, 1962, pp. 443-458.

[23] Braga, B.P.F., Yeh, W.W-G., Becker, L. and Barros, M.T.L. (1991), *Stochastic Optimization of Multiple-Reservoir-System Operation,* Jour-

nal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 117, No. 4, July/August 1991, pp. 131-147.

[24] Burn, D.H., Venema, H.D. and Simonović, S.P. (1991), *Risk-Based Performance Criteria for Real-Time Reservoir Operation,* Canadian Journal of Civil Engineering, Vol. 18, 1991, pp. 36-42.

[25] Burton, C.M. (1988), *Management of the River Murray During Periods of Extended Drought,* A Thesis for the Degree of Master of Engineering Science. Department of Civil Engineering, University of Adelaide, December 1988.

[26] Calabrese, G. (1947), *Generating Reserve Capacity Determined by the Probability Method,* AIEEE Transactions, Vol. 66, September 1947, pp. 1439-1450.

[27] Can, E.K. and Houck, M.H. (1984), *Real-Time Reservoir Operations by Goal Programming,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 110, No. 3, July 1984, pp. 297-309.

[28] Carnell, J. (1986), *Short Term Demand Forecasting,* in Water Demand Forecasting : Proceedings of a Workshop, edited by Gardiner, V. and Herrington, P., Geo Books, Norwich, United Kingdom, 1986, pp. 77-85.

[29] Carver, P.H. and Boland, J.J. (1980), *Short- and Long-run Effects of Price on Municipal Water Use,* Water Resources Research, Vol. 16, No. 4, August 1980, pp. 609-616.

[30] Cassuto, A.E. and Ryan, S. (1979), *Effect of Price on the Residential Demand for water within an Agency,* Water Resources Bulletin, Vol. 15, No. 2, April 1979, pp. 345-353.

[31] Charnes, A., Cooper, W.W. and Symonds, G.H. (1958), *Cost Horizons and Certainty Equivalents : An Approach to Stochastic Programming of Heating Oil,* Management Science, Vol. 4, No. 3, April 1958, pp. 235-263.

[32] Changchit, C. and Terrell, M.P. (1989), *CCGP Model for MultiObjective Reservoir Systems,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 115, No. 5, September 1989, pp. 658-671.

[33] Chu, W.S. and Yeh, W.W-G. (1978), *A Non-Linear Programming algorithm for real-time hourly Reservoir Operations,* Water Resources Bulletin, Vol. 14, No. 5, October 1978, pp. 1048-1063.

[34] Chung, F.I., Archer, M.C., and DeVries, J.J. (1989), *Network Flow Algorithm Applied to California Aqueduct Simulation,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 115, No. 2, March 1989, pp. 131-147.

[35] Claps, P., Rossi, F. and Vitale, C. (1993), *Conceptual-Stochastic Modelling of Seasonal Runoff Using Autoregressive Moving Average Models and Different Scales of Aggregation,* Water Resources Research, Vol. 29, No. 8, August 1993, pp. 2545-2559.

[36] Clark, E.J. (1950), *New York Control Rules,* Journal of the American Water Works Association, Vol. 42, No. 9, September 1950, pp. 823-827.

[37] Clark, E.J. (1956), *Impounding Reservoirs,* Journal of the American Water Works Association, Vol. 48, No. 4, April 1956, pp. 349-354.

[38] Clark, W.C. (1980), *Witches, Floods and Wonder Drugs : Historical Perspectives on Risk Management,* in 'Societal Risk Assessment: How Safe is Safe Enough ?' , edited by R.C. Schwing and W.A. Albers Jnr., (Proc. General Motors Symposium on Risk Assessment, Warren, Michigan, October 1979.) Plenum Press, New York, 1980, pp. 287-313.

[39] Clark, R.D.S. (1992), *Multi-Objective Management of Urban Stormwater in Adelaide,* International Symposium on Urban Stormwater Management, Sydney, Australia, 4-7 February, 1992, The Institution of Engineers, Australia, Preprints of Papers pp. 298-304.

[40] Clarke, K.F., Page, C. and Brew, J.S. (1980), *Reliability of Water Resources,* Journal of the Institute of Water Engineers and Scientists, Vol. 34, No. 1, January 1980, pp. 61-73.

[41] Close, A. (1994), Engineer, Systems Modelling, Murray Darling Basin Commission, Canberra, Australian Capital Territory, Personal Communication, 1994.

[42] Cochran, R. and Cotton, A.W. (1985), *Municipal Water Demand Study, Oklahoma City and Tulsa, Oklahoma,* Water Resources Research, Vol. 21, No. 7, July 1985, pp. 941-943.

[43] Codner, G.P. (1978), *A Dynamic Programming Approach to the Optimisation of a Complex Urban Water Supply Scheme,* Technical Paper No. 47, Australian Water Resources Council Australian Government Publishing Service, Canberra, 1979.

[44] Cohen, B.L. and Lee, I.S. (1979), *A Catalogue of Risks,* Health Physics, Vol. 6, No. 36, June 1979, pp. 707-722.

[45] Colorni, A., and Fronza, G. (1976), *: Reservoir Management via Reliability Programming,* Water Resources Research, Vol. 12, No. 1, February 1976, pp. 85-88.

[46] Crawley, P.D. and Dandy, G.C. (1989), *Optimal Operating Policies for Multiple Reservoir Systems,* Australian Water Research Advisory Council Research Project P86/15, Final Report, Department of Primary Industries and Energy, Australian Government Publishing Service, Canberra, September 1989.

[47] Crawley, P.D. and Dandy, G.C. (1989), *Optimum Reservoir Operating Policies including Salinity Considerations,* Hydrology and Water Resources Symposium 1989, Christchurch, New Zealand, 23-30 November, 1989, The Institution of Engineers, Australia, Preprints of Papers pp. 289-293.

[48] Crawley, P.D. (1990), *Optimal Operating Policies for Multiple Reservoir Systems,* A Thesis for the Degree of Master of Engineering Science. Department of Civil Engineering, University of Adelaide, April 1990.

[49] Crawley, P.D. and Dandy, G.C. (1992), *Reliability Cost Tradeoffs for Multiple Reservoir Systems,* Research Report. Department of Civil Engineering, University of Adelaide, August 1992.

[50] Crawley, P.D. and Dandy, G.C. (1993), *Optimal Operation of a Multiple Reservoir System,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 119, No. 1, January/February 1993, pp. 1-15.

[51] Crawley, P.D. and Dandy, G.C. (1993), *A Headworks Optimisation Model Incorporating a Graphical User Interface,* WATERCOMP '93, Melbourne, Australia, 30 March - 1 April, 1993, The Institution of Engineers, Australia, Preprints of Papers pp. 319-324.

[52] Crawley, P.D. (1993), *Reliability Assessment of Components of the Metropolitan Adelaide Headworks Pumping System,* E&WS Library Reference No. 93/25 (Unrestricted).

[53] Cullinane, M.J. (1985), *Reliability Evaluation of Water Distribution System Components,* in Hydraulics and Hydrology in the Small Computer Age - Volume 1, Waldrop, W. (Editor), American Society of Civil Engineers, New York, 1985, pp. 353-358.

[54] Cullinane, M.J. (1989), *Determining Availability and Reliability for Water Distribution Systems,* in Reliability Analysis of Water Distribution

Systems, Chapter 7, edited by L.M. Mays, American Society of Civil Engineers, New York, 1989, pp. 190-246.

[55] Cunnane, C. (1978), *Unbiased Skewed Positions - A Review,* Journal of Hydrology, Vol. 37, No. 3/4, May 1978, pp. 205-222.

[56] Dalkey, N. and Helmer, O. (1963), *An Experimental Application of the DELPHI Method to the Use of Experts,* Management Science, Vol. 9, No. 3, April 1963, pp. 458-467.

[57] Damelin, E. , Shamir, U. and Arad, N. (1972), *Engineering and Economic Evaluation of the Reliability of Water Supply,* Water Resources Research, Vol. 8, No. 4, August 1972, pp. 861-877.

[58] Dandy, G.C. , Burton, C.M. and Neave, G. (1986), *Towards Development of Optimum Operating Policies for the Adelaide Headworks System,* Hydrology and Water Resources Symposium 1986, Brisbane 25-27 November, 1986, The Institution of Engineers, Australia, Preprints of Papers pp. 129-134.

[59] Dandy, G.C. (1987), *A study of the Factors which affect Residential Water Consumption in Adelaide. Final Report.* Research Report No. R78, Department of Civil Engineering, University of Adelaide, March 1987.

[60] Dandy, G.C. (1988), *Developing Reliability-Cost Tradeoffs for a Multiple Reservoir System,* Hydrology and Water Resources Symposium 1988, Canberra 1-3 February, 1988, The Institution of Engineers, Australia, Preprints of Papers pp. 197-200.

[61] Dandy, G.C. and Simpson, A.R. (Editors) (1989), *Proceedings of the National Workshop on Planning and Management of Water Resource Systems : Risk and Reliability,* Adelaide, 23-25 November, 1988, Conference Series No. 17, Australian Water Resources Council, Australian Government Publishing Service, Canberra, July 1989.

[62] Dandy, G.C. and Crawley, P.D. (1990), *Optimisation of Cost and Salinity in Reservoir Operations,* in Water Resource Systems Application, Simonović, S.P. et al (Editors), University of Manitoba, Canada, June 1990, pp. 452-461.

[63] Dandy, G.C. and Crawley, P.D. (1991), *Application of Linear Programming to the Planning and Operation of Reservoir Systems with Water Quality Considerations,* Pre-Symposium Workshop entitled "Real-World Applications of Optimisation Techniques to Water Resources Management", International Hydrology and Water Resources Symposium 1991, The Institution of Engineers, Australia, Perth, W.A., Australia, Oct. 1991.

[64] Dandy, G.C. and Baker, T.J. (1991), *Use of Multisite Time Series Models for Flow Forecasting,* Hydrology and Water Resources Symposium 1991, Perth 2-4 October, 1991, The Institution of Engineers, Australia, Preprints of Papers pp. 787-792.

[65] Dandy, G.C. and Crawley, P.D. (1992), *Optimum Operation of a Multiple Reservoir System Including Salinity Effects,* Water Resources Research, Vol. 28, No. 4, April 1992, pp. 979-990.

[66] Dandy, G.C. (1992), *Assessing the Economic Cost of Restrictions on Outdoor Water Use,* Water Resources Research, Vol. 28, No. 7, July 1992, pp. 1759-1766.

[67] Danielson, L.E. (1979), *An Analysis of Residential Demand for Water using Micro Time-Series Data,* Water Resources Research, Vol. 15, No. 4, August 1979, pp. 763-767.

[68] Dantzig, G.B. (1955), *Linear Programming under Uncertainty,* Management Science, Vol. 1, No. 3, April-July 1955, pp. 197-206.

[69] Dantzig, G.B. and Madansky, A. (1961), *On the solution of two-state linear programs under uncertainty,* in Proceedings of the Fourth Berkeley

Symposium on Mathematical Statistics and Probability , pp. 165-17, University of California Press, Berkeley, California, 1961.

[70] Datta, B. and Houck, M.H. (1984), *A Stochastic Optimization Model for Real-Time Operation of Reservoirs Using Uncertain Forecasts,* Water Resources Research, Vol. 20, No. 8, August 1984, pp. 1039-1046.

[71] Davis D.R. (1977), Comment on *Bayesian Generation of Synthetic Streamflows,* by Vicens G.J., Rodríguez-Iturbe, I. and Schaake J.C. (1975), Water Resources Research, Vol. 13, No. 5, October 1977, pp. 853-854.

[72] DeGroot, M.H. (1974), *Reaching a Consensus,* Journal of the American Statistical Association, Vol. 69, No. 345, March 1974, pp. 118-121.

[73] Diment, G.A. (1991), *The Wide Use of a General Headworks and Resources Simulation Model - REALM,* Pre-Symposium Workshop entitled "Real-World Applications of Optimisation Techniques to Water Resources Management", International Hydrology and Water Resources Symposium 1991, The Institution of Engineers, Australia, Perth, W.A., Australia, Oct. 1991.

[74] Divi, R. et al. (1979), *Non-Linear Programming applied to the optimal operation of a Hydro-Electric Power System,* Tenth International Symposium of the Mathematical Programming Society, The Mathematical Programming Society, Montreal, Quebec, 1979.

[75] Dorfman, R. (1962), *Mathematical Models : The multi-structure approach,* in Design of Water-Resource Systems : new techniques for relating economic objectives, engineering analysis, and governmental planning, Chapter 13, edited by A. Maass et al., Harvard University Press, Cambridge, Massachusetts, 1962, 494-539.

[76] Druce, D.J. (1990), *Incorporating Daily Flood Control Objectives Into a Monthly Stochastic Dynamic Programming Model for a Hydroelectric*

Complex, Water Resources Research, Vol. 26, No. 1, January 1990, pp. 5-11.

[77] Duncan, H. and Kesari, N. (1989), *Planning and Management of Melbourne's Water Supply System,* in the Proceedings of the National Workshop on Planning and Management of Water Resource Systems : Risk and Reliability, Dandy, G.C. and Simpson, A.R. (Editors), University of Adelaide, Adelaide, 23-25 November, 1988, Conference Series No. 17, Australian Water Resources Council, Australian Government Publishing Service, Canberra, July 1989, pp. 304-311.

[78] Dunnette, M.D., Campbell, J. and Jaasted, L. (1963), *The Effect of Group Participation on Brainstorming Effectiveness for Two Industrial Samples,* Journal of Applied Psychology, Vol. 47, No. 1, February 1963, pp. 30-37.

[79] Dupacova, J. (1980), *Water Resource System Modeling using stochastic programming models,* in Recent Results in Stochastic Programming , edited by P. Kall and A. Prékopa, Springer-Verlag, New York, 1980, 121-133.

[80] Duckstein, L., Plate, E.J. and Benedini, M. (1987), *Water Engineering Reliability and Risk : A System Framework,* in Engineering Reliability and Risk in Water Resources, edited by Duckstein, L. and Plate, E.J., NATO ASI Series, Series E : Applied Sciences, Vol. 124, Martinus Nijhoff, Dordrecht, 1987, pp. 1-20.

[81] Duckstein, L., Shrestha, B.P. and Stakhiv, E.Z. (1991), *Multicriterion Risk and Reliability Analysis in Hydrologic System Design and Operation,* in Water Resources Engineering Risk Assessment, edited by Ganoulis, J., NATO ASI Series, Series G : Ecological Sciences, Vol. 29, Springer-Verlag, Berlin, 1991, pp. 363-385.

[82] Dziegielewski, B., and Crews, J.E. (1986), *Minimizing the Cost of Coping with Droughts : Springfield, Illnois,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 112, No. 4, October 1986, pp. 419-438.

[83] Edwards, W., Phillips, L.D., Hays, W.L. and Goodman, B.C. (1968), *Probabilistic Information Processing Systems : Design and Evaluation,* IEEE Transactions on Systems, Science and Cybernetics, Vol. SSC-4, 1968, pp. 248-265.

[84] Einhorn, H.J. (1972), *Expert Measurement and Mechanical Combination,* Organizational Behaviour and Human Performance, Vol. 7, No. 1, February 1972, pp. 86-106.

[85] Einhorn, H.J. and Hogarth, R.M. (1975), *Unit Weighting Schemes for Decision Making,* Organizational Behaviour and Human Performance, Vol. 13, No. 2, April 1975, pp. 171-192.

[86] Estes, W.K. (1976), *The Cognitive Side of Probability Learning,* Psychological Review, Vol. 83, No. 1, January 1976, pp. 37-64.

[87] Evanson, D.E. and Mosley, J.C. (1970), *Simulation Optimisation techniques for multi-basin water resource planning,* Water Resources Bulletin, Vol. 6, No. 5, September-October 1970, pp. 725-736.

[88] Engineering and Water Supply Department (1971), *Water Treatment for Metropolitan Adelaide,* Coudrey Offset Press, Adelaide, 1971.

[89] Engineering and Water Supply Department (1980), *River Torrens Flood Mitigation Study : Technical report No. 3 - Flood Mitigation Alternatives and Solution Optimisation,* E&WS Library Reference No. 80/73 (Unrestricted).

[90] Engineering and Water Supply Department (1984), *Trends in Water Consumption of Metropolitan Adelaide 1969 - 1983,* E&WS Library Reference No. 84/7 (Unrestricted).

[91] Engineering and Water Supply Department (1992), *Minutes forming Enclosure to EWS 5517/92,* E&WS Internal Project Docket.

[92] Fiering, M.B. (1967), *Streamflow Synthesis,* Harvard University Press, Cambridge, Massachusetts, 1967.

[93] Fiering, M.B. (1968), *Schemes for Handling Inconsistent Matrices,* Water Resources Research, Vol. 4, No. 2, April 1968, pp. 291-297.

[94] Fiering, M.B. and Jackson, B.B. (1971), *Synthetic Streamflows,* American Geophysical Union, Water Resources Monograph 1.

[95] Fiering, M.B. (1976), *The Role of Systems Analysis in Water Program Developments,* Natural Resources Journal, Vol. 16, October 1976, pp. 759-771.

[96] Fiering, M.B. (1982), *A Screening Model to Quantify Resilience,* Water Resources Research, Vol. 18, No. 1, February 1982, pp. 27-32.

[97] Fiering, M.B. (1982), *Alternative Indices of Resilience,* Water Resources Research, Vol. 18, No. 1, February 1982, pp. 33-39.

[98] Fischhoff, B. (1975), *Hindsight ≠Foresight : The Effect of Outcome Knowledge on Judgement and Uncertainty,* Journal of Experimental Psychology : Human Perception and Performance, Vol. 1, No. 3, August 1975, pp. 288-299.

[99] Fujiwara, O. and Ganesharajah, T. (1993), *Reliability Assessment of Water Supply Systems With Storage and Distribution Networks,* Water Resources Research, Vol. 29, No. 8, August 1993, pp. 2917-2924.

[100] Fussell, J.B. (1975), *Computer Aided Fault Tree Construction,* in Reliability and Fault Tree Analysis, edited by R.E. Barlow and J.B. Fussell, Society for Industrial and Applied Mathematics, Philadelphia, 1975, pp. 37-56.

[101] Galambos, J. (1978), *The Asymptotic Theory of Extreme Order Statistics,* John Wiley and Sons, New York, 1978.

[102] Ganoulis, J. (1991), *Water Resources Engineering Risk Assessment,* edited by Ganoulis, J., NATO ASI Series, Series G : Ecological Sciences, Vol. 29, Springer-Verlag, Berlin, 1991.

[103] Genest, C. and Zidek, J.V. (1986), *Combining Probability Distributions : A Critique and Annotated Bibliography,* Statistical Science, Vol. 1, No. 1, February 1986, pp. 114-148.

[104] Georgakakos, A.P. and Marks, D.H. (1987), *A New Method for the Real-Time Operation of Reservoir Systems,* Water Resources Research, Vol. 23 No. 7, July 1987, pp. 1376-1390.

[105] Georgakakos, A.P. and Yao, H. (1989), *Extended Linear Quadratic Gaussian Control : Further Extensions,* Water Resources Research, Vol. 25 No. 2, February1989, pp. 191-201.

[106] Georgakakos, A.P. and Yao, H. (1993), *New Control Concepts for Uncertain Water Resources Systems : Theory,* Water Resources Research, Vol. 29 No. 6, June 1993, pp. 1505-1516.

[107] Georgakakos, A.P. (1993), *Operational Tradeoffs in Reservoir Control,* Water Resources Research, Vol. 29 No. 11, November 1993, pp. 3801-3819.

[108] Giles, J.E. and Wunderlich W.O. (1981), *Weekly Multipurpose Planning Model for TVA Reservoir system,* Journal of the Water Resources Plan-

ning and Management Division, American Society of Civil Engineers, Vol. 108, No. WR2, October 1981, pp. 495-511.

[109] Gibbs, K.C. (1978), *Price Variable in Residential Water Demand Models,* Water Resources Research, Vol. 14 No. 1, January 1978, pp. 15-18.

[110] Gordon, W.J.J. (1961), *Synectics,* Harper and Row, New York, 1961.

[111] Goulter, M.H. and Tai, F.K. (1985), *Practical Implications in the Use of Stochastic Dynamic Programming for Reservoir Operations,* Water Resources Bulletin, Vol. 21, No. 1, February 1985, pp. 65-74.

[112] Green, A.E. and Bourne, A.J. (1972), *Reliability Technology,* Wiley-Interscience, London ; New York, 1972.

[113] Grunewald, O.C., Haan, C.T., Debertin, D.L. and Carey, D.I. (1976), *Rural Residential Water Demand : An Econometric and Simulation Analysis,* Water Resources Bulletin, Vol. 12, No. 5, October 1976, pp. 951-961.

[114] Grygier, J.C. and Stedinger, J.R. (1985), *Algorithms for Optimising Hydropower System Operation,* Water Resources Research, Vol. 21 No. 1, January 1985, pp. 1-11.

[115] Goulter, I.C. and Tai, M.H. (1985), *Practical Implications in the Use of Stochastic Dynamic Programming for Reservoir Operation,* Water Resources Bulletin, Vol. 21, No. 1, February 1985, pp. 65-74.

[116] Gove P.B. (1986), *Websters Third International Dictionary of the English Language Unabridged,* Merriam-Webster Inc., Springfield, Massachusetts, USA, 1986.

[117] Guildford, J.P. (1967), *The Nature of Human Intelligence,* McGraw-Hill, New York, 1967.

[118] Gumbel, E.J. (1958), *Statistics of Extremes,* Columbia University Press, New York, 1958.

[119] Haasl, D.F. (1965), *Advanced Concepts in Fault Tree Techniques,* in Proc. System Safety Symp., Seattle, 1965 The Boeing Company, Seattle, 1965.

[120] Hall, J.D., Ringlee, R.J. and Wood, A.J. (1968), *Frequency and Duration Methods for Power System Reliability Calculations : I - Generation System Model,* IEEE Transactions Power Apparatus and Systems, Vol. PAS-87, No. 9, September 1968, pp. 1787-1796.

[121] Hall, W.A., Tauxe, G.W. and Yeh W.W-G. (1969), *An alternative procedure for the optimisation of operations for planning with multiple river, multiple purpose systems,* Water Resources Research, Vol. 5, No. 6, December 1969, pp. 1367-1372.

[122] Hanke, S.H., and de Maré, L. (1982), *Residential Water Demand : A Pooled Time Series, Cross-Section Study of Malmö, Sweden,* Water Resources Bulletin, Vol. 18, No. 4, August 1982, pp. 621-626.

[123] Hanks, P., and Wilkes, G. A. (1986), *Collins dictionary of the English language : an extensive coverage of contemporary international and Australian English,* 2nd. Edition, Collins, Sydney, 1986.

[124] Hanmer, W. (1972), *Handbook of System and Product Safety,* Prentice-Hall, Englewoods Cliffs, New Jersey, 1972.

[125] Hansen, R.D., and Narayanan, R. (1981), *A Monthly Time Series Model of Municipal Water Demand,* Water Resources Bulletin, Vol. 17, No. 4, August 1981, pp. 578-585.

[126] Hall, J.D., Ringlee, R.J. and Wood, A.J. (1968), *Frequency and Duration Methods for Power System Reliability Calculations : I - Generation System Model,* IEEE Transactions Power Apparatus and Systems, Vol. PAS-87, No. 9, September 1968, pp. 1787-1796.

[127] Halperin, H. and Adler, H.A. (1958), *Determination of Reserve Generating Capability,* AIEE Transactions (Power Apparatus and Systems), Vol. 77, August 1958, pp. 530-544.

[128] Harboe, R., Kularathna, M.D.U.P and Bogardi, J.J. (1991), *Stochastic Dynamic Programming for Operation of Two Reservoirs,* Hydrology and Water Resources Symposium 1991, Perth 2-4 October, 1991, The Institution of Engineers, Australia, Preprints of Papers pp. 793-796.

[129] Harr, M.E. (1987), *Reliability-Based Design in Civil Engineering,* McGraw-Hill Book Company, New York, 1987.

[130] Hashimoto, T. Stedinger, J.R. and Loucks, D.P. (1982), *Reliability, Resiliency and Vulnerability Criteria For Water Resource System Performance Evaluation,* Water Resources Research, Vol. 18, No. 1, February 1982, pp. 14-20.

[131] Hastings, N.A.J. (1973), *Dynamic Programming with Management Applications,* Crane Russal and Company, New York, 1973.

[132] Heidari, M., Chow, V.T., Kokotović and Meredith, D.D. (1971), *Discrete Differential Dynamic Programming Approach to water Resources Systems Optimisation,* Water Resources Research, Vol. 7, No. 2, April 1971, pp. 273-283.

[133] Henley, E.J., and Kumamoto, H. (1992), *Probabilistic Risk Assessment - Reliability Engineering, Design and Analysis,* IEEE Press, New York, 1992.

[134] Hobbs, B.F. (1985), *Reliability Analysis of Urban Water Supply,* in Computer Applications in Water Resources , edited by H.C. Tomo, American Society of Civil Engineers, New York, 1985, pp. 1229-1238.

[135] Hobbs, B.F. (1985), *Reliability Analysis of Water System Capacity,* in Hydraulics and Hydrology in the small computer Age - Volume 1, Wal-

drop, W. (Editor), American Society of Civil Engineers, New York, 1985, pp. 341-346.

[136] Hobbs, B.F., Beim, G.K. and Gleit, A. (1987), *Reliability Analysis of Power and Water Supply Systems,* in Strategic Planning in Energy and Natural Resources, Lev, B. et al. (Editors), North Holland, Amsterdam, 1987, 105-120.

[137] Hobbs, B.F. and Beim, G.K. (1988), *Analytical Simulation of Water System Capacity Reliability 1. Modified Frequency-Duration Analysis,* Water Resources Research, Vol. 24, No. 9, September 1988, pp. 1431-1444.

[138] Hobbs, B.F. and Beim, G.K. (1989), *An Overview of Integrated Water Supply System Availability,* in Reliability Analysis of Water Distribution Systems, Chapter 10, edited by L.M. Mays, American Society of Civil Engineers, New York, 1989, pp. 299-339.

[139] Hobbs, B.F. (1989), *Integrated Water Supply System Evaluation : Two Methodologies,* in Reliability Analysis of Water Distribution Systems, Chapter 11, edited by L.M. Mays, American Society of Civil Engineers, New York, 1989, pp. 340-388.

[140] Hogan, A.J., Morris, J.G. and Thompson, H.E. (1981), *Decision Problems under Risk and Chance Constrained Programming : Dilemmas in the Transition,* Management Science, Vol. 27, No. 6, June 1981, pp. 698-716.

[141] Hogarth, R.M. (1987), *Judgement and Choice : The Psychology of Decision,* John Wiley and Sons, Chichester, 1987.

[142] Hooper, E.R., Georgakokas, A.P. and Lettenmaier, D.P. (1991), *Optimal Stochastic Operation of Salt River Project, Arizona,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 117, No. 5, September/October 1991, pp. 566-587.

[143] Hora, S.C., Hora, J.A. and Dodd, N.G. (1992), *Assessment of Probability Distributions for Continuous Random Variables : A Comparison of the Bisection and Fixed Value Methods,* Organizational Behaviour and Human Decision Processes, Vol. 51, No. 1, February 1992, pp. 133-155.

[144] Hora, S.C. (1992), *Acquisition of Expert Judgement : Examples from Risk Assessment,* Journal of Energy Engineering, American Society of Civil Engineers, Vol. 118, No. 2, August 1992, pp. 136-148.

[145] Houck, M.H. and Cohon, J.L. (1978), *Sequential Explicitly Stochastic Linear Programming Models : A Proposed Method for Design and Management of Multipurpose Reservoir Systems,* Water Resources Research, Vol. 14, No. 2, April 1978, pp. 161-169.

[146] Houck, M.H. (1979), *A chance-constrained Optimisation Model for Reservoir Design and Operation,* Water Resources Research, Vol. 15 No. 5, August 1979, pp. 1011-1016.

[147] Houck, M.H., and Datta, B. (1981), *Performance Evaluation of a Stochastic Optimisation Model for Reservoir Design and Management with Explicit Reliability Criteria,* Water Resources Research, Vol. 17, No. 4, October 1981, pp. 827-832.

[148] Howe, C.W. and Linaweaver, F.P. (1967), *The Impact of Price on Residential Water Demand and Its Relation to System Design and Price Structure,* Water Resources Research, Vol. 3, No. 1, March 1967, pp. 13-32.

[149] Howe, C.W. (1982), *The Impact of Price on Residential Water Demand : Some New Insights,* Water Resources Research, Vol. 18, No. 4, August 1982, pp. 713-716.

[150] Huang, W-C., Harboe, R. and Bogardi, J.J. (1991), *Testing Stochastic Dynamic Programming Models Conditioned on Observed or Forecasted*

Inflows, Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 117, No. 1, January/February 1991, pp. 28-36.

[151] Hudson, H.E. and Roberts, W.J. (1955), *1952-1955 Illnois Drought with Special reference to Impounding Reservoir Design,* Illnois State Water Survey Bulletin, No. 43, Illnois, 1955.

[152] Hydrologic Engineering Center (1981), *HEC-3 Reservoir System Analysis for Conservation, Programmer's Manual,* U.S. Army Corps of Engineers, Davis, California, 1981.

[153] Hyman, R. (1961), *On Prior Information and Creativity,* Psychological Reports, Vol. 9, 1961, pp. 151-161.

[154] Ikebuchi, S. (1990), *Study on Characteristics of Drought by FTA (Fault Tree Analysis),* Simonović, S.P. et al (Editors), University of Manitoba, Canada, June 1990, pp. 187-196.

[155] Institute of Electrical and Electronics Engineers (1981), *Guide for General Principles of Reliability Analysis of Nuclear Power Generating Station Protection Systems,* (An American National Standard) ANSI N41-4-1976, IEEE Std. 352-1975 (Revision of IEEE Std. 352-1972), New York, 1975.

[156] Institution of Professional Engineers New Zealand. President's Task Committee on Professional Practice and Risk (1984), *Engineering risk: report of the President's Task Committee on Professional Practice and Risk.,* Institution of Professional Engineers New Zealand, Wellington, N.Z, 1984.

[157] Jain, S.K., Yoganarasimhan, G.N. and Seth, S.M. (1992), *A Risk-Based Approach for Flood Control Operation of a Multiple Reservoir System,* Water Resources Bulletin, Vol. 28, No. 6, December 1992, pp. 1037-1043.

[158] Johnson, D.M. (1972), *Systematic Introduction to the Psychology of Thinking,* Harper and Row, New York, 1972.

[159] Johnson, S.A., Stedinger, J.R. and Stachus, K. (1991), *Heuristic Operating Policies for Reservoir System Simulation,* Water Resources Research, Vol. 27, No. 5, May 1991, pp. 673-685.

[160] Jordan, W.E. and Marshall, G.C. (1972), *Failure Mode, Effects and Criticality Analysis,* Annual Reliability and Maintainability Symposium, San Francisco, California, 1972.

[161] Kalbfleisch, J.G. (1985), *Probability and statistical inference,* 2nd ed., Springer-Verlag, New York, 1985.

[162] Kall, P. (1982), *Stochastic Programming,* European Journal of Operational Research, Vol. 10, No. 2, June 1982, pp. 125-130.

[163] Kaplan, S. and Garrick, B. (1981), *On the Quantitative Definition of Risk,* Risk Analysis, Vol. 1, No. 1, 1981, pp. 11-27.

[164] Karamouz, M., and Houck, M.H. (1987), *Comparison of Stochastic and Deterministic Dynamic Programming for Reservoir Operating Rule Generation,* Water Resources Bulletin, Vol. 23, No. 1, February 1987, pp. 1-9.

[165] Karamouz, M., Houck, M.H. and Delleur, J.W. (1992), *Optimisation and Simulation of Multiple Reservoir Systems,* Journal of the Water Resources Planning and Management, American Society of Civil Engineers, Vol. 118, No. 1, January/February 1992, pp. 71-81.

[166] Karamouz, M., and Vasiliadis, H.V. (1992), *Bayesian Stochastic Optimisation of Reservoir Operation Using Uncertain Forecasts,* Water Resources Research, Vol. 28, No. 5, May 1992, pp. 1221-1232.

[167] Katzman, M.T. (1977), *Income and Price Elasticities of Demand for water in Developing Countries,* Water Resources Bulletin, Vol. 13, No. 1, February 1977, pp. 47-55.

[168] Kelman, J., Stedinger, J.R., Cooper, L.A., Hsu, E. and Yuan, S-Q. (1990), *Sampling Stochastic Dynamic Programming Applied to Reservoir Operation,* Water Resources Research, Vol. 26, No. 3, March 1990, pp. 447-454.

[169] Kher, L.K. and Sarooshian, S. (1986), *Identification of Water Demand Models From Noisy Data,* Water Resources Research, Vol. 22, No. 3, March 1986, pp. 322-330.

[170] Krishan, B. and Pauperas, J. (1991), *Decision Theory, Failure Tolerance Analysis, and Quantitative Risk Analysis - Synergistic Application to the Space Station Freedom WP-2 Program,* in The Analysis, Communication, and Perception of Risk, edited by B.J. Garrick and W.C. Gekler, Advances in Risk Analysis, Volume 9, Plenum Press, New York, 1991, pp. 495-506.

[171] Kuczera, G. and Diment, G. (1988), *A general Water Supply System Model : WASP,* Journal of the Water Resources Planning and Management, American Society of Civil Engineers, Vol. 114, No. 4, July 1988, pp. 365-382.

[172] Kuczera, G. (1989), *Fast Multireservoir Multiperiod Linear Programming Models,* Water Resources Research, Vol. 25, No. 2, February 1989, pp. 169-176.

[173] Kuczera, G. (1991), *Use of Generalized Headworks Simulation Models as Planning Tools,* Pre-Symposium Workshop entitled "Real-World Applications of Optimisation Techniques to Water Resources Management", International Hydrology and Water Resources Symposium 1991, The Institution of Engineers, Australia, Perth, W.A., Australia, Oct. 1991.

[174] Labadie, J.W., Bode, D.A and Pineda, A.M. (1986), *Network Model for Decision Support in Municipal Water Raw Water Supply,* Water Resources Bulletin, Vol. 22, No. 6, December 1986, pp. 927-940.

[175] Larson, R.E. (1968), *State Incremental Dynamic Programming,* Elsevier, New York, 1968.

[176] Lauria, D.T. (1983), *Introduction to AWWA Economics Research Committee Report,* Journal of the American Water Works Association, Vol. 75, No. 1, January 1983, pp. 4-5.

[177] Lauria, D.T. (1983), *Research Needs for Capacity Planning,* Journal of the American Water Works Association, Vol. 75, No. 1, January 1983, pp. 14-19.

[178] Lawley, H.G. (1974), *Operability Study and Hazards Analysis,* Chemical Engineering Progress, Vol. 70, No. 4, April 1974, pp. 45-56.

[179] Lawley, H.G. (1976), *Size up Plant Hazards this way,* Hydrocarbon Processing, Vol. 55, No. 4, April 1976, pp. 247-261.

[180] Lee, E.S. and Waziruddin, S. (1970), *Applying Gradient Projection and Conjugate Gradient to the Optimum Operation of Reservoirs,* Water Resources Bulletin, Vol. 6, No. 5, October 1970, pp. 713-724.

[181] Lees, F.P. (1980), *Loss Prevention in the Process Industries - Hazard Identification, Assessment and Control (Volume 1),* Butterworths, London, 1980.

[182] Lees, F.P. (1980), *Loss Prevention in the Process Industries - Hazard Identification, Assessment and Control (Volume 2),* Butterworths, London, 1980.

[183] Lindner, M.A., Samad, F.A. and Howell, D.T. (1980), *The Use of Synthetic Hydrology in Decision Making in a Complex River Valley,* Hydrology and Water Resources Symposium 1980, Adelaide, Australia, 4-6 November, 1980, The Institution of Engineers, Australia, Preprints of Papers pp. 119-127.

[184] Linstone, H.A. and Turoff, M. (1975), *The Delphi Method : Techniques and Applications,* Addison-Wesley, Reading, Massachusetts, 1975.

[185] Loaiciga, H.A. and Mariño, M.A. (1986), *Risk Analysis for Reservoir Operation,* Water Resources Research, Vol. 22, No. 4, April 1986, pp. 483-488.

[186] Loganathan, G.V. and Bhattacharya, D. (1990), *Goal-Programming Techniques for Optimal Reservoir Operations,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 116, No. 6, November/December 1990, pp. 820-838.

[187] Loucks, D.P. (1968), *Computer Models for Reservoir Regulation,* Journal of the Sanitary Engineering Division, American Society of Civil Engineers, Vol. 94, No. SA4, August 1968, pp. 657-669.

[188] Loucks, D.P. (1970), *Some Comments on Linear Decision Rules and Chance Constraints,* Water Resources Bulletin, Vol. 6, No. 2, April 1970, pp. 668-671

[189] Loucks, D.P. and Falkson, L.M. (1970), *A Comparison of some Dynamic, Linear and Policy Iteration Methods for Reservoir Operation,* Water Resources Bulletin, Vol. 6, No. 3, 1970, pp. 384-400.

[190] Loucks, D.P., Stedinger, J.R. and Haith, D.A. (1981), *Water Resource Systems Planning and Analysis,* Prentice-Hall, Englewood Cliffs, N.J., 1981.

[191] Lyman, R.A. (1992), *Peak and Off-Peak Residential Water Demand,* Water Resources Research, Vol. 28, No. 9, September 1992, pp. 2159-2167.

[192] MacGregor, D., Lichtenstein, S. and Slovic, P. (1988), *Structuring Knowledge Retrieval : An Analysis of Decomposing Quantitative Judge-*

ment, Organizational Behaviour and Human Decision Processes, Vol. 42, No. 3, December 1988, pp. 303-323.

[193] Maidment, D.R., and Parzen, E. (1984), *Cascade Model of Monthly Municipal Water Use,* Water Resources Research, Vol. 20, No. 1, January 1984, pp. 15-23.

[194] Maidment, D.R., and Parzen, E. (1984), *Time Patterns of Water Use in Six Texas Cities,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 110, No. 1, January 1984, pp. 90-106.

[195] Maidment, D.R., Miaou, S-P. and Crawford, D.D. (1985), *Transfer Function Models of Daily Urban Water Use,* Water Resources Research, Vol. 21, No. 4, April 1985, pp. 425-432.

[196] Maidment, D.R. and Miaou, S-P. (1986), *Daily Water Use in Nine Cities,* Water Resources Research, Vol. 22, No. 6, June 1986, pp. 845-851.

[197] Manne, A.S. (1962), *Product-Mix Alternatives : Flood Control, Electric Power and Irrigation,* International Economic Review, Vol. 3, No. 1, January 1962, pp. 30-59.

[198] Mariño, M.A., and Mohammadi, B. (1983), *Reservoir Management : A Reliability Programming Approach,* Water Resources Research, Vol. 19, No. 3, June 1983, pp. 613-620.

[199] Martin, Q.W. (1987), *Optimal Daily Operation of Surface-Water Systems,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 113, No. 4, July 1987, pp. 453-470.

[200] Martin, W.E., and Kulakowski, S. (1991), *Water Price as a Policy Variable in Managing Urban Water Use : Tuscon, Arizona,* Water Resources Research, Vol. 27, No. 2, February 1991, pp. 157-166.

[201] Matalas, N.C (1967), *Mathematical Assessment of Synthetic Hydrology,* Water Resources Research, Vol. 3, No. 4, Fourth Quarter 1967, pp. 937-945.

[202] McLeod, A.I. and Hipel, K.W. (1978), *Simulation Procedures for Box-Jenkins models,* Water Resources Research, Vol. 14, No. 5, October 1978, pp. 969-975.

[203] McMahon, T.A. and Mein, R.G. (1986), *River and Reservoir Yield,* Water Resources Publications, Littleton, Colorado, 1986.

[204] Metropolitan Water Authority (1985), *Domestic Water Use in Perth, Western Australia,* Metropolitan Water Centre, Leederville, Western Australia, June 1985.

[205] Miaou, S-P. (1990), *A Class of Time Series Urban Water Demand Models with Nonlinear Climatic Effects,* Water Resources Research, Vol. 26, No. 2, February 1990, pp. 169-178.

[206] Miaou, S-P. (1990), *A Stepwise Time Series Regression Procedure for Water Demand Model Identification,* Water Resources Research, Vol. 26, No. 9, September 1990, pp. 1887-1897.

[207] Misra, K.B. (1992), *Reliability Analysis and Prediction - A Methodology Oriented Treatment,* Elsevier, Amsterdam, 1992.

[208] Mohan, S. and Keskar, J.B. (1991), *Optimisation of Multipurpose Reservoir System Operation,* Water Resources Bulletin, Vol. 27, No. 4, August 1991, pp. 621-629.

[209] Moncur, J.E.T. (1987), *Urban Water Pricing and Drought Management,* Water Resources Research, Vol. 23, No. 3, March 1987, pp. 393-398.

[210] Moncur, J.E.T. (1989), *Drought Episodes Management : The Role of Price,* Water Resources Bulletin, Vol. 25, No. 3, June 1989, pp. 499-505.

[211] Moreau, D.H. (1991), *A Risk-Based Model for Drought Contingency Planning,* Water Resources Bulletin, Vol. 27, No. 1, February 1991, pp. 117-123.

[212] Morgen, W.D. and Smolen, J.C. (1976), *Climatic Indicators in the Estimation of Municipal Water Demand,* Water Resources Bulletin, Vol. 12, No. 3, June 1976, pp. 511-518.

[213] Morris, J.R. and Jones, C.V. (1980), *Water for Dencer : An Analysis of the Alternatives,* Environmental Defense Fund Inc., Denver, Colorado, 1980.

[214] Moy, W-S., Cohon, J.L. and ReVelle, C.S. (1986), *A Programming Model for the Analysis of the Reliability, Resilience and Vulnerability of a Water Supply Reservoir,* Water Resources Research, Vol. 22, No. 4, April 1986, pp. 489-498.

[215] Mujumdar, P.P. and Vedula, S. (1992), *Performance Evaluation of an Irrigation System under some Optimal Operating Policies,* Hydrological Sciences Journal, Vol. 37 No. 1, February 1992, pp. 13-26.

[216] Mukjerjee, D. and Mansour, N. (1991), *A Bivariate Stochastic Approach to Estimate Reliability of a Pumped Storage Reservoir,* Journal of Hydrology, Vol. 125, No. 3/4, July 1991, pp. 293-310.

[217] Nardini, A., Piccardi, C. and Soncini-Sessa, R. (1992), *On the Integration of Risk Aversion and Average-Performance Optimisation in Reservoir Control,* Water Resources Research, Vol. 28, No. 2, February 1992, pp. 487-497.

[218] Narayanan, R., Larson, D.T. and Hughes, T.C. (1985), *Effectiveness of Drought Policies for Municipal Water Management,* Water Resources Bulletin, Vol. 21, No. 3, 1985, pp. 407-416.

[219] Nayak, S., and Arora, S.R. (1971), *Optimal Capacities for a multireservoir system using the Linear Decision Rule,* Water Resources Research, Vol. 7, No. 3, 1971, pp. 485-498.

[220] Nielsen, D.S. (1971), *The Cause-Consequence Diagram Method as a Basis for Quantitative Accident Analysis,* Dutch Atomic Energy Commission, Report RISØ, M. 1374, Denmark, 1971.

[221] Nielsen, N.M., Vick, S.G. and Hartford, D.N.D. (1994), *Risk Analysis in British Columbia,* International Water Power and Dam Construction, Vol. 46, No. 3, March 1994, pp. 35-40.

[222] National Safety Council (1979), *Accident Facts,* Washington, D.C., 1979.

[223] Ng, W.S., and Kuczera, G. (1993), *Incorporating Demand Uncertainty in Water Supply Headworks Simulation,* Water Resources Research, Vol. 29, No. 2, February 1993, pp. 469-477.

[224] Nopmongcol, P., and Askew, A.J. (1976), *Multi-level Incremental Dynamic Programming,* Water Resources Research, Vol. 12, No. 6, December 1976, pp. 1291-1297.

[225] Novak, J.T. (1975), *Planning for Emergencies at Water Utilities,* Journal of the American Water Works Association, Vol. 67, No. 4, April 1975, pp. 164-167.

[226] Osborn, A.F. (1953), *Applied Imagination,* Schribner, New York, 1953.

[227] Palmer, R.N., Smith, J.A., Cohon, J.A. and ReVelle, C.S. (1982), *Optimal Operation of a Multiple Reservoir System,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 108, No. WR1, March 1982, pp. 47-66.

[228] Palencia, L.C. (1988), *Residential Water Demand in Metro Manila,* Water Resources Bulletin, Vol. 24, No. 2, April 1988, pp. 275-279.

[229] Palmer, R.N., and Lettenmaier, D.P. (1983), *The Use of Screening Models in Determining Water Supply Reliability,* Civil Engineering Systems, Vol. 1, No. 1, January 1983, pp. 15-22.

[230] Pegram, G.G.S., and James, W. (1972), *Multilag Multivariate Autoregressive Model for the Generation of Operational Hydrology,* Water Resources Research, Vol. 8, No. 4, August 1972, pp. 1074-1076.

[231] Pereira, M.V.F., and Pinto, L.M.V.G. (1985), *Stochastic Optimisation of a Multireservoir Hydroelectric System : A Decomposition Approach,* Water Resources Research, Vol. 21, No. 6, June 1985, pp. 779-792.

[232] Perera, B.J.C., and Codner, G.P. (1985), *Application of Stochastic Dynamic Programming to the operation of a Multiple Reservoir System,* Hydrology and Water Resources Symposium 1985, Sydney 14-16 May, 1985, The Institution of Engineers, Australia, Preprints of Papers pp. 29-33.

[233] Perera, B.J.C., and Codner, G.P. (1988), *A Combined Stochastic Dynamic Programming-Statistical Disaggregation Approach applied to Multiple Reservoir Systems,* Water Resources Management, Vol. 2, 1988, pp. 153-171.

[234] Petrakian, R., Haimes, Y.Y. and Stakhiv, E. (1989), *Risk Analysis of Dam Failure and Extreme Floods,* in Risk Analysis and Management of Natural and Man-made Hazards, edited by Y.Y. Haimes, and E. Stakhiv, 1987 Engineering Foundation Conference, American Society of Civil Engineers, 1989, pp. 81-123.

[235] Phillips, D.T., Ravindran, A. and Solberg, J.J. (1976), *Operations Research : Principles and Practice,* John Wiley and Sons, New York, 1976.

[236] Piccardi, C., and Soncini-Sessa, R. (1991), *Stochastic Dynamic Programming for Reservoir Optimal Control: Dense Discretization and Inflow*

Correlation Assumption made possible by Parralel Computing, Water Resources Research, Vol. 27, No. 5, May 1991, pp. 729-741.

[237] Powers, G.J., and Tompkins, F.C. (1974), *Fault Tree Synthesis for Chemical Processes,* American Institute of Chemical Engineers Journal, Vol. 20, No. 2, March 1974, pp. 376-387.

[238] Powers, G.J., Tompkins, F.C. and Lapp, S.A. (1975), *A Safety Simulation Language for Chemical Processes : A Procedure for Fault Tree Synthesis,* in Reliability and Fault Tree Analysis, edited by R.E. Barlow and J.B. Fussell, Society for Industrial and Applied Mathematics, Philadelphia, 1975, pp. 57-75.

[239] Prasifka, D.W. (1984), *Water Supply Management Enters a New Era,* Civil Engineering, Vol. 54, No. 3, March 1984, pp. 45-47.

[240] Prékopa, A. (1980), *Network planning using two-stage programming under uncertainty,* in Recent Results in Stochastic Programming , edited by P. Kall and A. Prékopa, Springer-Verlag, New York, 1980, pp. 215-237.

[241] Prince, G.M. (1969), *How to be a Better Meeting Chairman,* Harvard Business Review, Vol. 47, No. 1, January-February 1969, pp. 98-108.

[242] Pulkkinen, U. (1993), *Methods for Combination of Expert Judgements,* Reliability Engineering and System Safety, Vol. 40, 1993, pp. 111-118.

[243] Quimpo, R.G. and Shamsi, U.M. (1987), *An Overview of Integrated Water Supply System Availability,* in Hydraulic Engineering, edited by R.M. Ragan, American Society of Civil Engineers, New York, 1987, pp. 716-721.

[244] Recht, J.L. (1966), *Failure Mode and Effect.* National Safety Council, 1966.

[245] Reid, S.G. (1989), *Risk Assessment.* Research Report No. R591, School of Civil and Mining Engineering, The University of Sydney, February 1989.

[246] Reinhardt, K. (1991), *Critical Item Ranking for the Space Shuttle Main Engine,* in Risk Analysis - Prospects and Opportunities, edited by C. Zervos, Advances in Risk Analysis, Volume 8, Plenum Press, New York, 1991, pp. 337-346.

[247] Reny, D.A., Mackowiac, D.P. and Stallman, R.M. (1991), *Risk Assessment and Risk Management for an Experimental Nuclear Waste Processing Facility,* in Risk Analysis - Prospects and Opportunities, edited by C. Zervos, Advances in Risk Analysis, Volume 8, Plenum Press, New York, 1991, pp. 347-355.

[248] ReVelle, C., Joeres, E. and Kirkby, W. (1969), *The Linear Decision Rule in Reservoir Management and Design, 1, Development of the Stochastic Model,* Water Resources Research, Vol. 5, No. 4, 1969, pp. 767-777.

[249] ReVelle, C. and Kirkby, W. (1970), *The Linear Decision Rule in Reservoir Management and Design, 2, Performance Optimisation,* Water Resources Research, Vol. 6, No. 4, August 1969, pp. 1033-1044.

[250] Reznicek, K.K. and Simonović, S.P. (1970), *An Improved Algorithm for Hydropower Optimisation,* Water Resources Research, Vol. 26, No. 2, February 1990, pp. 189-198.

[251] Ringlee, R.J. and Wood, A.J. (1969), *Frequency and Duration Methods for Power System Reliability Calculations : II - Demand Model and Capacity Reserve Model,* IEEE Transactions Power Apparatus and Systems, Vol. PAS-88, No. 9, April 1969, pp. 375-388.

[252] Roberts, N.H., Vesely, W.E., Haasl, D.F and Goldberg, F.F (1981), *Fault Tree Handbook,* NUREG, 0492, U.S. Nuclear Regulatory Commission, January 1981.

[253] U.S. Nuclear Regulatory Commission (1981), *Fault Tree Handbook - NUREG/CR-0492,* U.S. Nuclear Regulatory Commission, January 1981.

[254] Roland, H.E., and Moriarty, B. (1990), *System Safety Engineering and Management,* John Wiley and Sons, Inc.

[255] Rosenthal, R.E. (1981), *A Non-Linear Network flow algorithm for maximisation of benefits in a Hydro-Electric Power System,* Operations Research, Vol. 29, No. 4, July-August 1981, pp. 763-786.

[256] Rubinstein, R.Y. (1981), *Simulation and the Monte Carlo Method,* Wiley Series in Probability and Mathematical Statistics, John Wiley and Sons, New York, 1981.

[257] Rubinstein, J., and Ortolano, L. (1984), *Water Conservation and Capacity Expansion,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 110, No. 2, April 1984, pp. 220-237.

[258] Sabet, M.H., Coe, J.Q., Ramirez, H.M., and Ford, D.T. (1985), *Optimal Operation of California Aqueduct,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 111, No. 2, April 1985, pp. 222-237.

[259] Sabet, M..H. and Coe, J.Q. (1986), *Models for Water and Power Scheduling for the California State Water Project,* Water Resources Bulletin, Vol. 22, No. 4, August 1986, pp. 587-596.

[260] Sabet, M.H. and Creel, C.L. (1991), *Network Flow Modelling of Oroville Complex,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 117, No. 3, May/June 1991, pp. 301-320.

[261] Sabet, M.H. and Creel, C.L. (1991), *Model Aggregation for the California State Water Project,* Journal of Water Resources Planning and Manage-

ment, American Society of Civil Engineers, Vol. 117, No. 5, September/October 1991, pp. 549-565.

[262] Sachs, L. and translated by Reynarowych, Z. (1982), *Applied Statistics : A Handbook of Techniques,* Springer-Verlag, New York, 1982.

[263] Salas, J.D. and Pegram, G.G.S. (1977), *A Seasonal Multivariate Multilag Autoregressive Model in Hydrology,* A paper presented at the Proceedings of the Third International Symposium on Theoretical and Applied Hydrology, Colorado State University, Fort Collins, Colorado.

[264] Salas, J.D., and Obeysekera, J.T.B. (1992), *Conceptual Basis of Seasonal Streamflow Time Series Models,* Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol. 118, No. 8, August 1992, pp. 1186-1194.

[265] Salas, J.D. (1993), *Models for Data Generation in Hydrology : Multivariate Techniques,* in Stochastic Hydrology and its Use in Water Resources Systems Simulation and Optimisation, edited by Marco, J.B., Harboe, R. and Salas J.D., Kluwer Academic Publications, Dordrecht, 1993, pp. 75-95.

[266] Salas, J.D. and Abdelmohsen, M.W. (1993), *Initialisation for Generating Single-Site and Multisite Low-Order Periodic Autoregressive and Moving Average Processes,* Water Resources Research, Vol. 29, No. 6, June 1993, pp. 1771-1776.

[267] Sauter, D.M., Baldwin, C.J. and Dale, K.M. (1963), discussion of Halperin and Adler [127], AIEE Transactions (Power Apparatus and Systems), Vol. 77, August 1958, pp. 541-542.

[268] Shamir, U. and Howard, C.D.D. (1981), *Water Supply Reliability Theory,* Journal of the American Water Works Association, Vol. 73, July 1981, pp. 379-384.

[269] Shamir, U. and Howard, C.D.D. (1985), *Reliability and Risk Assessment for Water Supply,* in Computer Applications in Water Resources , edited by H.C. Tomo, American Society of Civil Engineers, New York, 1985, pp. 1218-1228.

[270] Sheedy, B.J. and Kesari, N. (1989), *Survey of Planning and Operational Policies and Procedures for Major Urban Headworks Systems in Australia,* in the Proceedings of the National Workshop on Planning and Management of Water Resource Systems : Risk and Reliability, Dandy, G.C. and Simpson, A.R. (Editors), University of Adelaide, Adelaide, 23-25 November, 1988, Conference Series No. 17, Australian Water Resources Council, Australian Government Publishing Service, Canberra, July 1989, pp. 74-105.

[271] Shelton, R.A. (1979), *Management of TVA reservoir systems,* A paper presented at the Proceedings of the National Workshop on Reservoir Systems Operation, University of Colorado, Boulder, August 13-17, 1979, 318-335.

[272] Simonović, S.P , and Mariño, M.A. (1980), *: Reliability Programming in Reservoir Management 1. Single Multipurpose Reservoir,* Water Resources Research, Vol. 16, No. 5, October 1980, pp. 844-848.

[273] Simonović, S.P , and Mariño, M.A. (1981), *: Reliability Programming in Reservoir Management 2. Risk-Loss Functions,* Water Resources Research, Vol. 17, No. 4, August 1981, pp. 822-826.

[274] Simonović, S.P , and Mariño, M.A. (1982), *: Reliability Programming in Reservoir Management 3. System of Multipurpose Reservoirs,* Water Resources Research, Vol. 18, No. 4, August 1982, pp. 735-743.

[275] Simonović, S.P., Venema, H.D. and Burn, D.H. (1992), *Risk-Based Parameter Selection for Short-Term Reservoir Operation,* Journal of Hydrology, Vol. 131, No. 1-4, February 1992, pp. 269-291.

[276] Simpson, J.A., and Weiner, E.S.C. (1989), *The Oxford English Dictionary,* 2nd. Edition, Clarendon Press, Oxford, 1989.

[277] Smith, J.A. (1988), *A Model of Daily Municipal Water Use for Short-Term Forecasting,* Water Resources Research, Vol. 24, No. 2, February 1988, pp. 201-206.

[278] Soliman, S.A. and Christensen, G.S. (1986), *Application of Functional Analysis to Optimisation of a Variable Head Multireservoir Power System for Long-Term Regulation,* Water Resources Research, Vol. 22, No. 6, June 1986, pp. 852-858.

[279] Srikanthan, R., McMahon, T.A., Codner, G.P. and Mein, R.G. (1983), *Application of some Multisite Streamflow Generation Models,* Civil Engineering Transactions, The Institution of Engineers, Australia, Vol. 26, 1983, pp. 272-279.

[280] Stedinger, J.R. and Taylor, M.R. (1982), *Synthetic Streamflow Generation : 2. Effect of Parameter Uncertainty,* Water Resources Research, Vol. 18, No. 4, August 1982, pp. 919-924.

[281] Stedinger, J.R., Sule, B.F. and Loucks, D.P. (1984), *Stochastic Dynamic Programming Models for Reservoir Operation Optimization,* Water Resources Research, Vol. 20, No. 11, November 1984, pp. 1499-1505.

[282] Stedinger, J.R., Pei, D. and Cohn, T.A. (1985), *A Condensed Disaggregation Model for Incorporating Parameter Uncertainty into Monthly Reservoir Simulations,* Water Resources Research, Vol. 21, No. 5, May 1985, pp. 665-675.

[283] Stewart, M.G. (1991), *Probabilistic Risk Assessment of Quality Control and Quality Assurance Measures in Structural Design,* IEEE Transactions on Systems, Man and Cybernetics, Vol. 21, No. 5, September-October 1991, pp. 1000-1007.

[284] Stockton, F.R. (1884), *The Lady or the Tiger and Other Stories,* Schribner's, New York, 1884.

[285] Strycharczyk, J.B. and Stedinger, J.R. (1987), *Evaluation of a "Reliability Programming" Reservoir Model,* Water Resources Research, Vol. 23, No. 2, February 1987, pp. 225-229.

[286] Tai, M..H. and Goulter, I.C. (1987), *A Stochastic Dynamic Programming based approach to the Operation of a Multi-Reservoir System,* Water Resources Bulletin, Vol. 23, No. 3, June 1987, pp. 371-377.

[287] Tao, T. and Lennox, W.C. (1991), *Reservoir Operations by Successive Linear Programming,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 117, No. 2, March/April 1991, pp. 274-280.

[288] Taylor, D.W., Berry, P.C. and Block, C.H. (1958), *Does Group Participation when using Brainstorming Facilitate or Inhibit Creative Thinking ?,* Administrative Science Quarterly, Vol. 3, March 1958, pp. 23-47.

[289] Tejada-Guibert, J.A., Johnson, S.A. and Stedinger, J.R. (1993), *Comparison of Two Approaches for Implementing Multireservoir Operating Policies Derived Using Stochastic Dynamic Programming,* Water Resources Research, Vol. 29, No. 12, December 1993, pp. 3969-3980.

[290] Thomas, H.A. and Fiering, M.B. (1962), *Mathematical Synthesis of Streamflow Sequences for the Analysis of River Basins by Simulation,* in Design of Water Resource Systems : new techniques for relating economic objectives, engineering analysis, and governmental planning, Chapter 12, edited by A. Maass et al., Harvard University Press, Massachusetts, 1962, pp. 459-493.

[291] Thomas, J.F. and Syme, G.J. (1988), *Estimating Residential Price Elasticity of Demand for Water : A Contingent Valuation Approach,* Water Resources Research, Vol. 24, No. 11, November 1988, pp. 1847-1857.

[292] Thomson J.R. (1987), *Engineering Safety Assessment,* Longman Scientific and Technical, Essex, England, 1987.

[293] Toebes, G.H. and Sheppard, A.A. (Editors) (1979), *Proceedings of the National Workshop on Reservoirs Systems Operation,* University of Colorado, Boulder, Colorado, 13-17 August, 1979, American Society of Civil Engineers, New York, 1979.

[294] Tomlinson, G. (1994), Senior Hydrographer, Hydrology Unit, Water Resources Group, Department of Environment and Natural Resources, Adelaide, South Australia, Personal Communication, 1994.

[295] Trezos, and Yeh, W.W-G. (1987), *Use of Stochastic Dynamic Programming Models for Reservoir Management,* Water Resources Research, Vol. 23, No. 6, June 1987, pp. 983-996.

[296] Turgeon, A. (1981), *Optimal Short-term scheduling of Hydro Plants from the principle of Progressive Optimality,* Water Resources Research, Vol. 17, No. 3, June 1981, pp. 481-486.

[297] Turgeon, A. (1981), *A decomposition method for the long-term scheduling of reservoirs in series,* Water Resources Research, Vol. 17, No. 6, December 1981, pp. 1565-1570.

[298] Tversky, A. and Kahneman, D. (1973), *Availability : A Heuristic for Judging Frequency and Probability,* Cognitive Psychology, Vol. 5, No. 2, September 1973, pp. 207-232.

[299] Uckun, S., Dawant, B.M. and Kawamura, K. (1991), *Uncertainty Management in Engineering Risk Assessment,* in The Analysis, Communication, and Perception of Risk, edited by B.J. Garrick and W.C. Gekler, Plenum Press, New York, 1991, pp. 87-92.

[300] Unny, T.E., Divi, R., Hinton, B., and Robert, A. (1981), *A Model for Real-Time Operation of a large Multi-Reservoir Hydroelectric System,* A

paper presented at the Proceedings of the International Symposium on Real-Time Operation of Hydrosystems, University of Waterloo, Waterloo, Ontario, Canada, June 24-26, 1981, 336-360.

[301] U.S. Nuclear Regulatory Commission (1975), *Reactor Safety Study - An Assessment of Accident Risks in U.S. Commercial Nuclear Power Plants - WASH-1400,* U.S. Nuclear Regulatory Commission, Washington, D.C., October 1975.

[302] U.S. Nuclear Regulatory Commission (1983), *PRA Procedures Guide,* NUREG, CR-2300, U.S. Nuclear Regulatory Commission, Washington, D.C., 1983.

[303] Valdés, J.B., Rodríguez-Iturbe, I. and Vicens G.J. (1977), *Bayesian Generation of Synthetic Streamflows - 2. The Multivariate Case,* Water Resources Research, Vol. 13, No. 2, April 1977, pp. 291-295.

[304] Vasiliadis, H.V., and Karamouz, M. (1994), *Demand-Driven Operation of Reservoirs using Uncertainty-Based Optimal Operating Policies,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 120, No. 1, January/February 1994, pp. 101-114.

[305] Vesely, W.E. (1984), *Engineering Risk Assessment Analysis,* in Technological Risk Assessment, edited by P.F. Ricci, L.A. Sagan, and G.G. Whipple, Martinus Nijhoff Publishers, The Hague, 1984, pp. 49-84.

[306] Vicens G.J., Rodríguez-Iturbe, I. and Schaake J.C. (1975), *Bayesian Generation of Synthetic Streamflows,* Water Resources Research, Vol. 11, No. 6, December 1975, pp. 827-838.

[307] Vick, S.G. and Bromwell, L.G. (1989), *Risk Analysis for Dam Design in Karst,* Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 115, No. 6, June 1989, pp. 819-835.

[308] Villemeur, A. (1992), *Reliability, Availability, Maintainability, and Safety Assessment - Volume 1 : Methods and Techniques,* John Wiley and Sons, New York, 1992.

[309] Wagner, J.M., Shamir, U. and Marks, D.H. (1988), *Water Distribution Reliability : Analytical Methods,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 114, No. 3, May 1988, pp. 253-275.

[310] Wagner, J.M., Shamir, U. and Marks, D.H. (1988), *Water Distribution Reliability : Simulation Methods,* Journal of Water Resources Planning and Management, American Society of Civil Engineers, Vol. 114, No. 3, May 1988, pp. 276-294.

[311] Walker, S. and Wyatt, T. (1987), *Use of A Dynamic Programming Technique for Optimizing Operation of a Regional Water Resource System,* Journal of the International Water Supply Association (Aqua), No. 5, October 1987, pp. 242-248.

[312] Wang, D. and Adams, B.J.. (1986), *Optimisation of Real-Time Reservoir Operations with Markov Decision Processes,* Water Resources Research, Vol. 22, No. 3, March 1986, pp. 345-352.

[313] Wasimi, S.A. and Kitanidis, P.K.. (1983), *Real-Time Forecasting and Daily Operation of a Multireservoir System During Floods by Linear Quadratic Gaussian Control,* Water Resources Research, Vol. 19, No. 6, December 1983, pp. 1511-1522.

[314] Weerahandi, S. and Zidek, J.V. (1983), *Elements of Multi-Bayesian Decision Theory,* The Annals of Statistics, Vol. 11, No. 4, December 1983, pp. 1032-1046.

[315] Weeraratne, J.R., Logan, L. and Unny, T.E. (1986), *Performance Evaluation of Alternate Policies Policies on Reservoir System Operation,* Canadian Journal of Civil Engineering, Vol. 13, 1986, pp. 203-212.

[316] Wets, R.J.B. (1966), *Programming under uncertainty : The equivalent convex program,* Journal on Applied Mathematics, Society for Industrial and Applied Mathematics, Vol. 14, No. 1, January 1966, pp. 89-105.

[317] Whitlach, E.E. and Martin, M.J. (1988), *Identification of Monthly Trends in Urban Water Use,* Water Resources Bulletin, Vol. 24, No. 1, February 1988, pp. 169-174.

[318] Willsie, R.H. and Pratt, H.L. (1974), *Water Use Relationships and Projection corresponding with Regional Growth,* Water Resources Bulletin, Vol. 10, No. 2, April 1974, pp. 360-371.

[319] Wilson, L. (1989), *Addition of a Climate Variable to the Howe and Linaweaver Western Sprinkling Equation,* Water Resources Research, Vol. 25, No. 6, June 1989, pp. 1067-1069.

[320] Wong, S.T. (1972), *A Model on Municipal Water Demand, A Case study of Northeastern Illnois,* Land Economics, Vol. 48, No. 1, February 1972, pp. 34-44.

[321] Wunderlich, W.O. (1979), *Planned enhancements of water management methods for the TVA reservoir system,* A paper presented at the Proceedings of the National Workshop on Reservoir Systems Operation, University of Colorado, Boulder, August 13-17, 1979, 336-360.

[322] Yakowitz, S.J. (1982), *Dynamic Programming Applications in Water Resources,* Water Resources Research, Vol. 18, No. 4, August 1982, pp. 673-696.

[323] Yamada, K. (1977), *Reliability Activities at Toyota Motor Company,* Reports of Statistical Application Research, Union of Japanese Scientists and Engineers, Vol. 24, No. 3, September 1977, pp. 22-39.

[324] Yamauchi, H. and Huang, W-Y. (1977), *Alternative Models for Estimating the Time Series Components of Water Consumption Data,* Water Resources Bulletin, Vol. 13, No. 3, June 1977, pp. 599-610.

[325] Yao, H. and Georgakakos, A.P. (1993), *New Control Concepts for Uncertain Water Resources Systems : Reservoir Management,* Water Resources Research, Vol. 29 No. 6, June 1993, pp. 1517-1525.

[326] Yeh, W.W-G. (1979), *Real-Time Reservoir Operation : The California Central Valley Project case study,* A paper presented at the Proceedings of the National Workshop on Reservoir Systems Operation, University of Colorado, Boulder, August 13-17, 1979, 217-227.

[327] Yeh, W.W-G., Becker, L., Toy, D., and Graves, A. (1980), *Central Arizona Project : Operations Model,* Journal of Water Resources, Planning and Management, American Society of Civil Engineers, Vol. 106, No. WR2, July 1980, pp. 521-540.

[328] Yeh, W.W-G. (1985), *Reservoir Management and Operations Models : A State-of-the-Art Review,* Water Resources Research, Vol. 21, No. 12, December 1985, pp. 1797-1818.

[329] Young, G.Y. and Pisano, W.C. (1968), *Operational Hydrology Using Residuals,* Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 94, July 1968, pp. 909-923.

[330] Young, R.A. (1973), *Price Elasticity of Demand for Municipal Water : A Case study of Tuscon Arizona,* Water Resources Research, Vol. 9, No. 4, August 1973, pp. 1068-1072.

Appendix A

HOMA Model Formulation

This appendix contains details of the HOMA model formulation described in Section 4.3 of Chapter 4. These details include the constraint equations, the pump cost curves, the storage vs. evaporation curves and the remaining system parameters.

A.1 Constraint Equations

This objective function used in HOMA is subject to a set of linear constraint equations, which can be written in the following generalised form :

$$\sum_{c=1}^{C_j} W_t^{jc} = 1 \tag{A.1}$$

for $\quad t = 1, 2, \ldots, T \; ; \; j = 1, 2, \ldots, J$

$$\sum_{c=1}^{C_j} \xi_t^{jc} W_t^{jc} = PT_t^j \tag{A.2}$$

for $\quad t = 1, 2, \ldots, T \; ; \; j = 1, 2, \ldots, J$

where,

PT_t^j = Volume pumped via pump station (j) during period (t)

ξ_t^j = Pump-cost 'volume' coefficient for pump station (j) during period (t)

$$PT_t^j \;=\; \sum_{n=1}^{N} P_t^{jn} \tag{A.3}$$

for t = 1, 2, ..., T ; j = 1, 2, ..., J

where,

P_t^{jn} = Volume pumped via pump station (j) to reservoir (n) during period (t)

$$\eta_t^n S_t^n \;=\; \mu_{t-1}^n S_{t-1}^n \;+\; I_t^n \;+\; \sum_{j=1}^{J} P_t^j \;+\; \sum_{o=1}^{N} T_t^{no} \;- \\ \sum_{o=1}^{N} T_t^{on} \;+\; \sum_{o=1}^{N} O_t^{no} \;-\; \sum_{o=1}^{N} O_t^{on} \;-\; F_t^n \;-\; E_t^n \tag{A.4}$$

for t = 1, 2, ..., T ; n = 1, 2, ..., N

where,

E_t^n = Evaporation constant for reservoir (n) during period (t)

F_t^n = System supply from reservoir (n) to its water filtration plant during period (t)

$= F_t^m$

$=$ Filtration plant (m) throughput during period (t)

I_t^n $=$ Catchment Inflow to reservoir (n) during period (t)

T_t^{no} $=$ Reservoir Transfer volume to reservoir (n) from reservoir (o) during period (t)

η,μ $=$ Reservoir-storage evaporation coefficients for reservoir (n) for period (t)

$$\sum_{o=1}^{N} O_t^{no} + \sum_{o=1}^{N} T_t^{no} + I_t^n \leq \theta_t^n \tag{A.5}$$

for $\quad t = 1, 2, \ldots, T \; ; \; n = 1, 2, \ldots, N$

where,

θ_t^n $=$ Transfer capacity into reservoir (n) during period (t)

$$S_t^n + Q_t^n \geq \kappa_t^n \tag{A.6}$$

for $\quad t = 1, 2, \ldots, T \; ; \; n = 1, 2, \ldots, N$

where,

κ_t^n $=$ Target storage level for reservoir (n) at the end of period (t)

$$S_t^n \leq \lambda_t^n \tag{A.7}$$

for $\quad t = 1, 2, \ldots, T \; ; \; n = 1, 2, \ldots, N$

where,

λ_t^n = Capacity of reservoir (n)
at the end of period (t)

Equations A.1 and A.2 define the piecewise linearised pump-cost curves for pumping water to the headworks system via the pump station (j) for each time period (t). Details of the derivation of these equations can be found in Section A.2.

The maximum volume that can be pumped via the pump station (j) during time period (t) is defined by the pump-cost 'volume' coefficient $\xi_t^{jC_j}$ as given in Equation A.2.

Equation A.3 describes the mass balance constraints for water pumped via each pump station (j) during time period (t).

Equation A.4 is the reservoir water balance equation for each reservoir (n) during the time period (t). This generalised equation ensures mass balance within each reservoir.

Equation A.5 defines the catchment inflow and transfer limitations for each reservoir. These equations are particularly relevant to the Adelaide headworks where a number of the reservoirs are off-stream.

Equation A.6 gives the target storage constraints for each reservoir. When $(S_t^n < \kappa_t^n)$ then $(Q_t^n > 0)$ where Q_t^n represents the volume of target shortfall for reservoir (n) during period (t). A penalty is paid on this target shortfall within the objective function. The inclusion of the parameters Q_t^n ensure that if the system can not physically achieve the required reservoir target storage levels, the linear program is not invalidated.

The reservoir storage capacities λ_t^n for each reservoir (n) and each time period (t) are given in Equation A.7.

A.2 Pump Cost Curves

Pumping from the River Murray to the metropolitan Adelaide headworks system can be carried out through 3 major pumping/pipeline systems. Each of these systems consist of a number of pumps that can be run individually or in parallel.

The pumps are electrically powered and the E&WS is charged for the power consumed by the Electricity Trust of South Australia (ETSA). The Electricity Trust has seasonal electricity tariff schedules comprising Off-peak, On-peak and Special On-peak periods for the winter, spring/autumn, summer seasons. The combination of the number of pumps and arrangement of the electricity tariff periods, results in a non-linear pump-cost curve. A typical example of these pump-cost curves is shown in Figure A.1. All combinations of pumps and electricity tariff periods are considered, from no pumps running to all pumps running continuously.

A lower bound to the series of points plotted is taken and the curve converted into a piece-wise linear function. These simplifications enable convex pumping-cost curves to be included in the linear programming formulation in the following manner.

The convex cost curve $\psi_t^j(PT_t^j)$, $j = 1, 2, \ldots, J$ is first approximated in a piecewise linear fashion, as shown in Figure A.1. This is accomplished by carefully selecting a number of break points, where the pumping-cost curve slope changes.

If we let the number of linear segments approximating the piecewise linear cost function $\psi_t^j(PT_t^j)$ be given as C_j for pump station j, we can define

$$\xi_t^{j1},\ \xi_t^{j2}, \ldots,\ \xi_t^{jC_j} \tag{A.8}$$

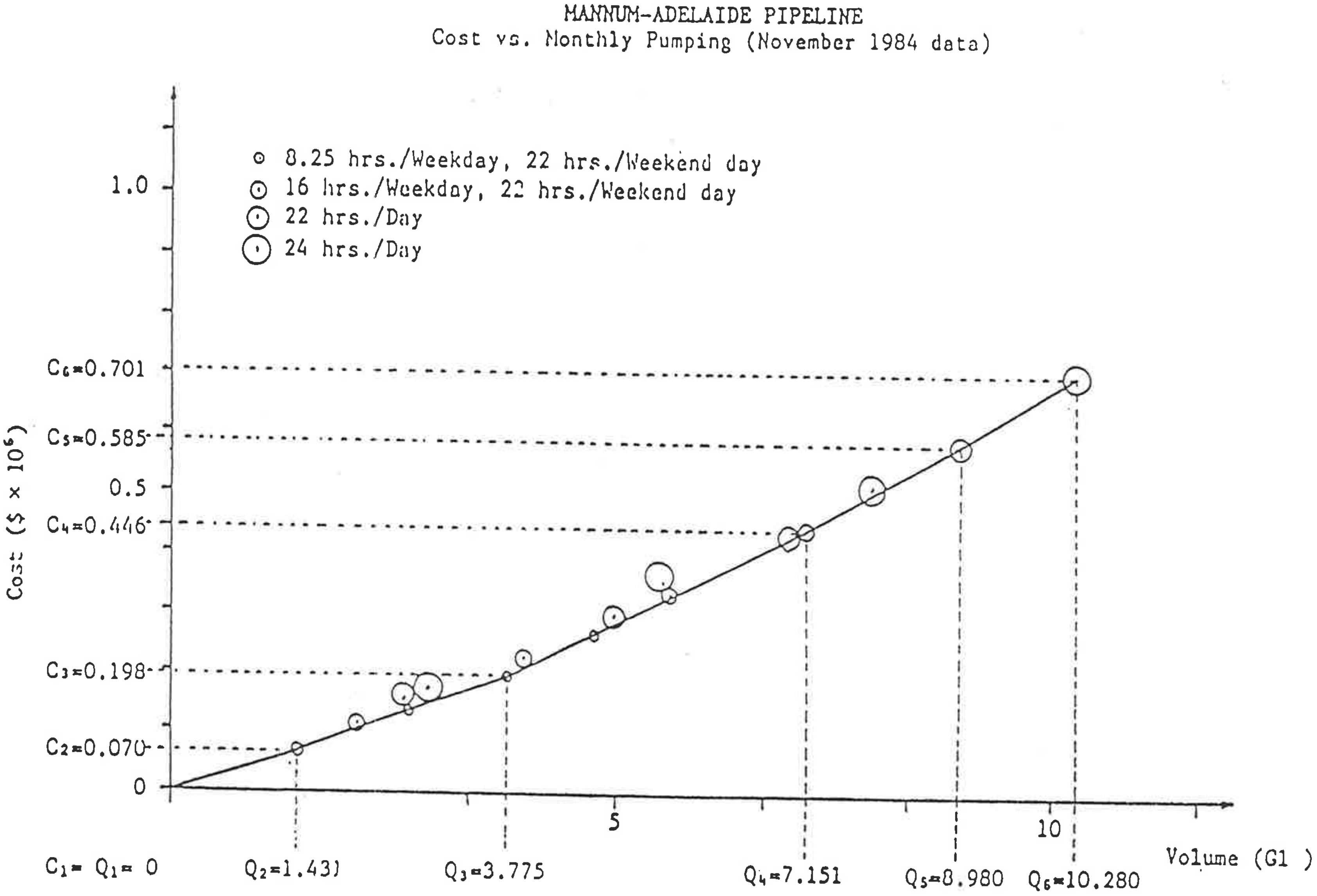


Figure A.1: Mannum - Adelaide Pump Cost Curve as the ascending values of PT_t^j where the piecewise linear cost function changes slope.

We next subdivide the volume pumped PT_t^j into a set of auxiliary components and define W_t^{jc} as the weight applying to the volume ξ_t^{jc} ie. if $W_t^{jc} = 1$, the volume pumped is ξ_t^{jc} ; if $W_t^{jc} = W_t^{j,c+1} = 0.5$ the volume pumped is midway between ξ_t^{jc} and $\xi_t^{j,c+1}$.

Then we can write :

$$PT_t^j \;=\; \sum_{c=1}^{C_j} \xi_t^{jc} W_t^{jc} \qquad \forall\, j,t \tag{A.9}$$

where

$$\sum_{c=1}^{C_j} W_t^{jc} \;=\; 1 \qquad \forall\, j,t \tag{A.10}$$

and

$$W_t^{jc} \;\geq\; 0 \qquad \forall\, j,t \tag{A.11}$$

then the nonlinear pumping-cost function can now be defined in terms of its piecewise linear approximation :

$$\psi_t^j(PT_t^j) \;=\; \sum_{c=1}^{C_j} \beta_t^{jc} W_t^{jc} \qquad \forall\, j,t \tag{A.12}$$

The problem has now been transformed into an ordinary linear programming problem that can be solved using a linear programming optimisation package in the conventional manner. Since the term $(\Delta\beta_t^{jc}/\Delta\xi_t^{jc})$ has a non-decreasing character, it is evident that in a cost minimisation problem the linear programming solution technique will select the cost variables in their correct order and no additional constraints are required.

A.3 Storage vs. Evaporation Curves

For each reservoir, evaporation losses can be plotted against storage volume for each time period and a linear regression applied to these plotted points. The resulting regression line takes the form of :

$$Er = \Upsilon + \Psi S \tag{A.13}$$

where,

Er = Reservoir Evaporation loss / mm depth of evaporation

S = Reservoir Storage

Ψ = The slope of the regression line

Υ = The 'y' intercept of the regression line

A typical example of the Evaporation loss vs. Storage volume curve with fitted regression line is shown in Figure A.2.

It has been assumed that the evaporation occurs at a constant rate during the time period considered. The total evaporation loss from reservoir (n) during a given time period (t) is then given as :

$$Ev_t^n = \left[\Upsilon^n + \Psi^n \frac{(S_t^n + S_{t-1}^n)}{2} \right] e_t^n \tag{A.14}$$

where,

e_t^n = Depth of evaporation for reservoir (n) during time period (t)

Ev_t^n = Total evaporation loss from reservoir (n) during time period (t)

The terms E_t^n, η_t^n and μ_t^n previously defined in the reservoir storage balance constraint equations in Section A.1 can then be calculated as :

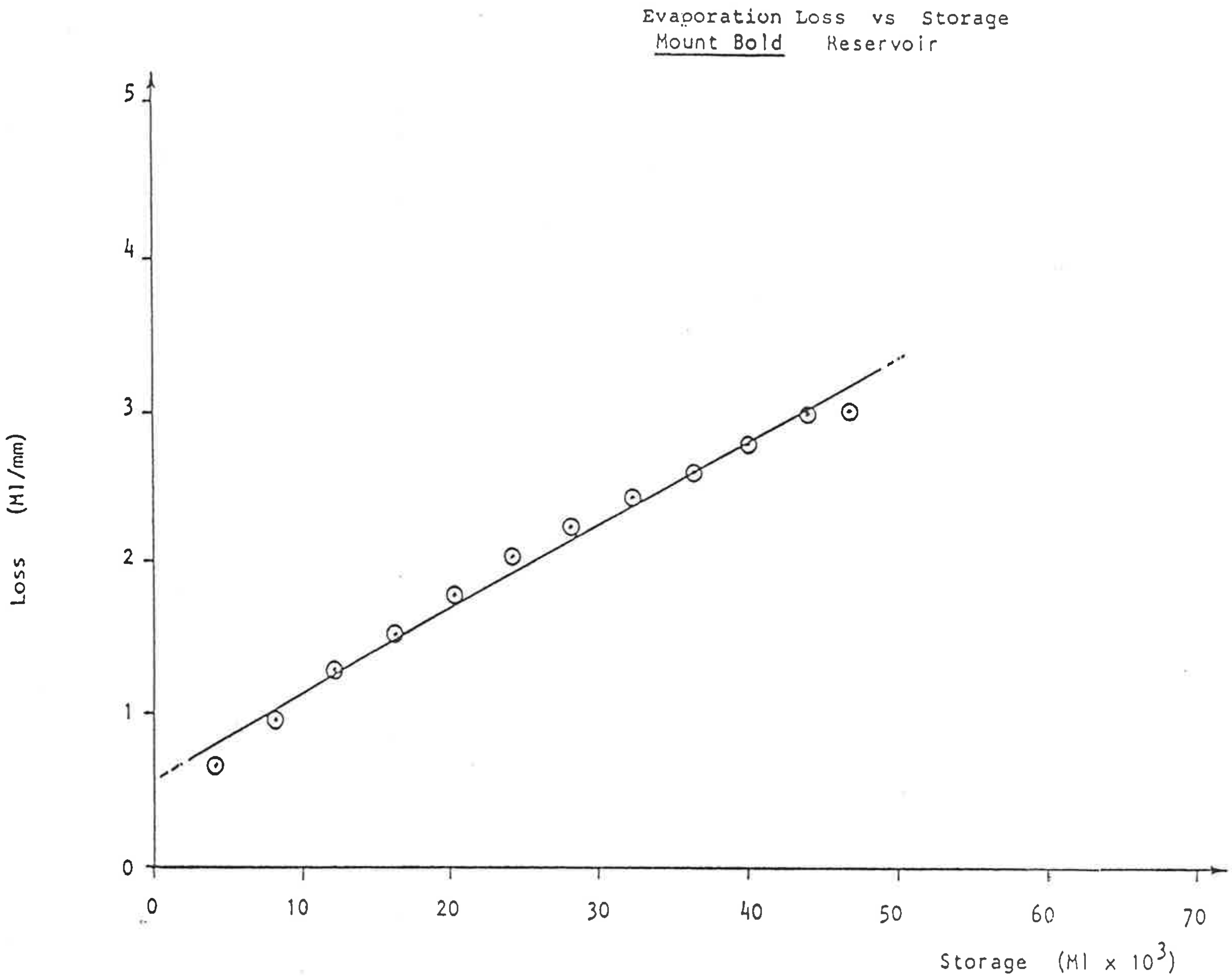


Figure A.2: Mount Bold Reservoir - Evaporation Loss vs. Storage curve

$$E_t^n = \Upsilon^n e_t^n \tag{A.15}$$

$$\eta_t^n = (1 + \frac{\Psi^n e_t^n}{2}) \tag{A.16}$$

$$\mu_t^n = (1 - \frac{\Psi^n e_t^n}{2}) \tag{A.17}$$

A.4 System Parameters

This section describes the remaining constraint equations in the linear program formulations for the southern and northern system models. These include :

- Constraints on the Mannum-Adelaide pipeline
- Constraints on the Swan Reach-Stockwell pipeline
- Constraints on certain reservoir intakes

The physical layout of the Mannum-Adelaide pipeline is described in Section 4.2.2. The constraint equations derived from this physical layout are described below. The capacity of the pipeline is not uniform along the length of the pipeline as shown in Figure 4.14. This non-uniformity has been modelled in the linear programming formulation using continuity constraint equations on the pipeline.

The generalised continuity equation for the Mannum-Adelaide pipeline can be written in the following form :

$$PT_t = \sum_{n=1}^{N} P_t^n + \sum_{k=1}^{K} F_t^m \qquad \forall\, t \tag{A.18}$$

where,

F_t^m = Filtration plant (m) throughput during period (t)

Constraints on the capacity of transfers from the pipeline into individual reservoirs are specified in the following generalised form :

$$P_t^n \leq \epsilon_t^n \qquad \forall\, j, t \tag{A.19}$$

where,

ϵ_t^n = Transfer capacity into reservoir (n) during time period (t)

The variable nature of the pipeline capacity along the length of pipe can not be described in a generalised form. For the Mannum-Adelaide pipeline the following limitations apply :

$$\begin{aligned}(\mathrm{PMAT})_t \;=\; & (\mathrm{PMAO})_t + (\mathrm{PMAW})_t + \\ & (\mathrm{PMAM})_t + (\mathrm{PMA4})_t \qquad \forall t\end{aligned} \tag{A.20}$$

$$(\mathrm{PMA4})_t + (\mathrm{PEM})_t = (\mathrm{PMA5})_t \qquad \forall t \tag{A.21}$$

$$(\mathrm{PMA5})_t = (\mathrm{PMALP})_t + (\mathrm{SUPAH})_t \qquad \forall t \tag{A.22}$$

and

$$(\mathrm{PMA4})_t \leq \tau_t \qquad \forall t \tag{A.23}$$

$$(\mathrm{PMA5})_t \leq \upsilon_t \qquad \forall t \tag{A.24}$$

where,

$(\mathrm{PMAT})_t$ = Total volume pumped in the Mannum-Adelaide pipeline during time period (t)

$(\mathrm{PMAO})_t$ = Volume supplied from the Mannum-Adelaide pipeline to on-line demands during time period (t)

$(\mathrm{PMAW})_t$ = Volume pumped through the Mannum-Adelaide pipeline to Warren reservoir during time period (t)

$(\mathrm{PMAM})_t$ = Volume pumped through the Mannum-Adelaide pipeline to the Torrens system during time period (t)

$(\mathrm{PMA4})_t$ = Volume pumped in the Mannum-Adelaide pipeline through section 4 during time period (t)

$(\mathrm{PEM})_t$ = Volume pumped from Millbrook reservoir to the Mannum-Adelaide pipeline during time period (t)

$(\mathrm{PMA5})_t$ = Volume pumped in the Mannum-Adelaide pipeline through section 5 during time period (t)

$(\mathrm{PMALP})_t$ = Volume pumped in the Mannum-Adelaide pipeline to the Little Para reservoir during time period (t)

$(\mathrm{SUPAH})_t$ = Supply from the Mannum-Adelaide pipeline to the Anstey Hill Water Filtration Plant during time period (t)

τ_t = Pipeline capacity for section 4 during time period (t)

υ_t = Pipeline capacity for section 5 during time period (t)

The constraints on the Swan Reach-Stockwell pipeline are of a slightly different nature to those on the Mannum-Adelaide pipeline. The physical layout of this pipeline system is shown in Figure 4.8. In order to keep the number of equations in the northern system linear programming formulation to a minimum, simplifications have been made to this pipeline system as shown in Figure 4.9.

It is assumed that all water pumped through the pipeline is placed into Warren reservoir. Water can then be transferred to South Para reservoir from Warren reservoir, limited only by the transfer capacity. Water supplied to the lower northern regions through the Swan Reach-Stockwell pipeline is assumed to have first been placed into Warren reservoir. In reality, when demands are high in the lower northern region, it is not possible to fully meet the supply from Warren reservoir because of hydraulic limitations on the pipeline.
This is modelled in the linear programming formulation by the following constraints :

$$(\mathrm{NDWR})_t \; - \; (\mathrm{PSR})_t \; \leq \; \omega_t \qquad \forall\, t \tag{A.25}$$

where,

$(\mathrm{NDWR})_t$ = Lower north demand met from the Swan Reach-Stockwell pipeline and Warren Reservoir during time period (t)

$(\mathrm{PSR})_t$ = Total volume pumped in the Swan Reach-Stockwell pipeline during time period (t)

ω_t = Hydraulic pipeline capacity limitations on the Warren trunk main during time period (t)

A number of the Adelaide metropolitan reservoirs were constructed as off-stream storages. Details of these reservoirs are given in Section 4.2. Intake into these reservoirs is limited by the capacity of the associated diversion structures, channels, pipes and tunnels. These limitations apply to instantaneous transfer rates and so various assumptions have been made on the appropriate limitations applying to the time periods considered. For certain of the reservoirs, the capacity of diversion structures (notably Millbrook reservoir) or the reservoir capacity and volume of natural intake (notably Barossa reservoir) enable these limitations to be ignored, however for the remaining off-stream storages notably Hope Valley and Happy Valley reservoirs these limitations need to be considered.

A characteristic of an off-stream storage is that it is constructed with very limited, if any, spillway capacity. If the capacity of the reservoir is reached, water is no longer diverted into the reservoir from the diversion structures, but instead allowed to spill at these locations.

The system schematic for Hope Valley reservoir is shown in Figure A.3.

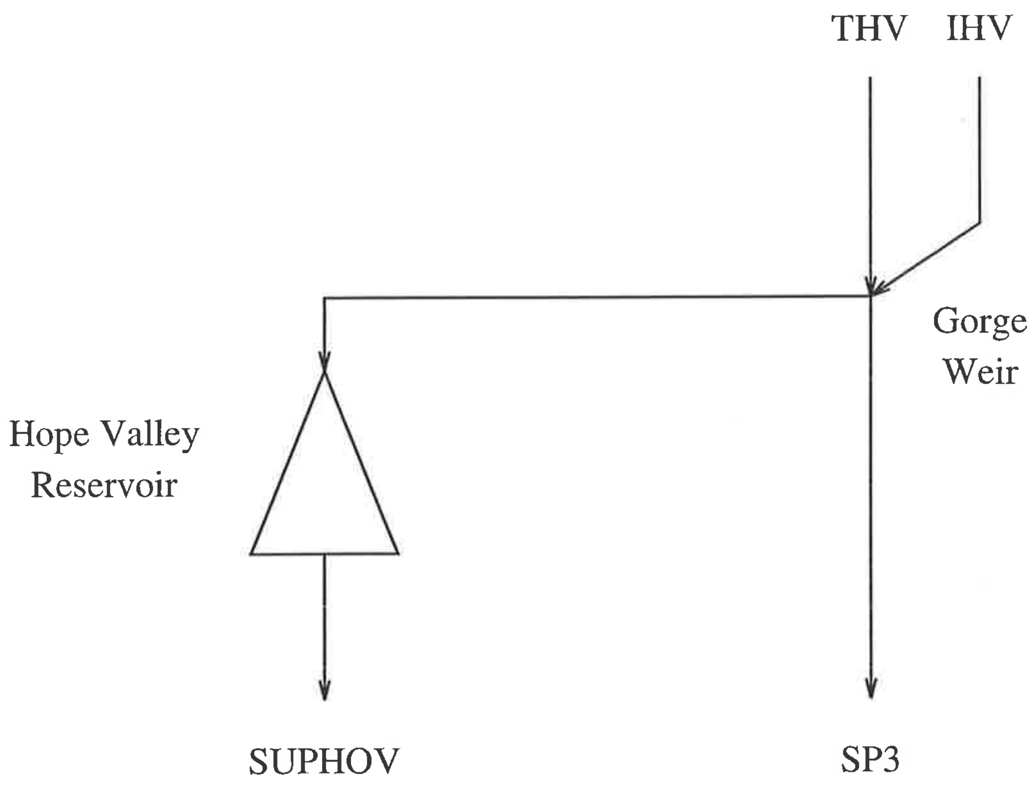


Figure A.3: The Hope Valley System Schematic

The particular constraint equations used to represent the Hope Valley Reservoir shown schematically in Figure A.3 and contained within the linear programming model are given by Equations A.26 and A.27.

$$(THV)_t + (IHV)_t - (SP3)_t \geq 0.0 \tag{A.26}$$

$$(\mathrm{THV})_t + (\mathrm{IHV})_t - (\mathrm{SP3})_t \leq 5.4 \qquad (A.27)$$

where,

$(\mathrm{IHV})_t$ = Specified catchment runoff volume downstream of Kangaroo Creek reservoir into Gorge Weir during time period (t)

$(\mathrm{SP3})_t$ = Release volume from Gorge Weir during time period (t)

$(\mathrm{SUPHOV})_t$ = Supply from the Hope Valley Reservoir to the Hope Valley Water Filtration Plant during time period (t)

$(\mathrm{THV})_t$ = Total Volume released from Kangaroo Creek reservoir to Gorge Weir during time period (t)

Equation A.26 ensures the volume of release from Gorge Weir during time period (t) does not exceed the sum of the catchment runoff and the release from Kangaroo Creek reservoir.
Equation A.27 limits the maximum volume of water that can be transferred from Gorge Weir to Hope Valley reservoir to the capacity of the Hope Valley aqueduct.

Appendix B

HOMA Model Data

B.1 HOMA data common to both Southern and Northern Systems

There are some data that is common to both southern and northern systems that will be presented first.

B.1.1 Shortfall Penalty Cost Coefficients

The objective function used in the HOMA model as presented in Section 4.3.1.1 of Chapter 4 in Equation 4.1 includes a shortfall penalty cost when a reservoir storage falls below the target storage level. The purpose of this shortfall penalty cost is to discourage but not prevent a solution containing reservoir storage levels below the target storage levels. During certain situations it is unavoidable that the reservoir storage levels will fall below these levels.
The penalty coefficient (α_t^n) has been set at 100 for all reservoirs (n) and time periods (t) in the runs undertaken using HOMA during the research presented in this thesis.

B.1.2 End of Period Benefit Coefficients

The objective function used in the HOMA model as presented in Section 4.3.1.1 of Chapter 4 in Equation 4.1 includes an 'end of period' benefit term for water held in storage at the end of the water year. The purpose of this 'end of period' benefit term is to maximise the end of period storages in each of the reservoirs. The benefit term (δ^n) has been set at 0.025 for all reservoirs (o) in the runs undertaken using HOMA during the research presented in this thesis.

B.1.3 Spill Penalty Cost Coefficients

The objective function used in the HOMA model as presented in Section 4.3.1.1 of Chapter 4 in Equation 4.1 includes a spill penalty cost. It is evident that if the cost of pumping is minimised and the the final storage level in the reservoirs is maximised through the inclusion of an 'end of period benefit' term that spillages from each reservoir will implicitly be minimised.

Consider however the system of reservoirs that are at full capacity and the forecast inflows for the next two months exceed the forecast demands. As far as the model is concerned there is no difference between a solution where all spills occur in the first month and the reservoir is drawn down only to be filled in the second month and a solution where the reservoir stays at full capacity for both months and the spills spread over the two months. In an operation sense the former practice of allowing the spill to occur during the first month only is unacceptable in the operation of the system. Thus in order to give credibility to the operation of the model, an appropriate set of very small penalty coefficients have been applied to the spill from the reservoirs.

The spill penalty coefficient (γ_t^o) has been set at 0.01 for all reservoirs (o) and time periods (t) in the runs undertaken using HOMA during the research presented in this thesis.

B.2 Southern System

The data applies to the southern system of the metropolitan Adelaide headworks system.

B.2.1 Reservoir Storage Capacities

The reservoir storage capacities used in the HOMA model runs are shown in Table B.1 below.

Reservoir	Capacity (GL)
Mount Bold	45.900
Happy Valley	12.700
Myponga	26.800

Table B.1: Southern System Reservoir Storage Capacities

B.2.2 Initial Reservoir Storage Levels

The starting reservoir storage levels for a HOMA model runs have been taken from the following start of year reservoir storage levels shown in Table B.2 below.

Reservoir	Storage (GL)
Mount Bold	45.900
Happy Valley	12.700
Myponga	25.301

Table B.2: Southern System - 'Start of Run' Reservoir Storage Levels

B.2.3 Indoor Water Use Demand Volumes

'Outdoor' water use restrictions have been considered for the southern Adelaide water supply system. In order to calculate the 'outdoor' water use component of the total demand on which restrictions can be imposed it is necessary to obtain a set of monthly 'indoor' water demands for each demand zone. The base monthly 'indoor' water demands obtained from the historical demand record for the last five years for the southern system are given in Table B.3.

Month	Happy Valley Zone Demand (SDHV) (GL)	Myponga Zone Demand (SDMY) (GL)	Encounter Bay Demand (SDEB) (GL)	Pipeline Online Demand (PMBOO) (GL)	Southern System Total Demand (GL)
July	3.500	0.170	0.050	0.300	4.020
August	3.500	0.170	0.050	0.300	4.020
September	3.500	0.170	0.050	0.300	4.020
October	3.500	0.170	0.050	0.300	4.020
November	3.500	0.170	0.050	0.300	4.020
December	3.500	0.170	0.050	0.300	4.020
January	3.500	0.170	0.050	0.300	4.020
February	3.500	0.170	0.050	0.300	4.020
March	3.500	0.170	0.050	0.300	4.020
April	3.500	0.170	0.050	0.300	4.020
May	3.500	0.170	0.050	0.300	4.020
June	3.500	0.170	0.050	0.300	4.020
TOTAL	42.000	2.040	0.600	3.600	48.240

Table B.3: Southern System - Assumed Indoor Water Use Volumes

B.2.4 Target Storage Levels

The target storage levels used in the operation of the southern metropolitan system comprise two components :

- Minimum Operating Level component
- Demand Storage Component

The minimum operating level component for the Mount Bold, Happy Valley and Myponga Reservoirs is shown in Table B.4.

Month	Mount Bold Reservoir (GL)	Happy Valley Reservoir (GL)	Myponga Reservoir (GL)
July	0.4	4.5	9.5
August	0.4	8.0	4.6
September	0.4	8.0	4.6
October	0.4	8.0	4.6
November	0.4	8.0	4.6
December	0.4	8.0	4.6
January	0.4	8.0	4.6
February	0.4	8.0	4.6
March	0.4	8.0	4.6
April	0.4	4.5	9.5
May	0.4	4.5	9.5
June	0.4	4.5	9.5

Table B.4: Southern System - Nominal Minimum Operating Levels

The demand storage components corresponding to the '8 weeks demand', '6 weeks demand', '4 weeks demand' and '2 weeks demand' criteria, described in Section 5.2.1.2 for the southern metropolitan system are shown in Tables B.5, B.6, B.7 and B.8 below.

Month	Mount Bold Reservoir (GL)	Happy Valley Reservoir (GL)	Myponga Reservoir (GL)
July	7.2	2.5	0.4
August	8.9	2.4	1.2
September	13.1	2.4	1.8
October	15.8	3.2	2.3
November	16.9	4.5	2.7
December	16.6	4.7	2.6
January	14.7	4.7	2.2
February	11.4	4.7	1.7
March	6.7	4.7	1.1
April	5.3	3.3	0.8
May	1.4	2.4	0.3
June	2.8	1.9	0.3

Table B.5: Southern System - '8 Weeks Demand' Target Storage Component

Month	Mount Bold Reservoir (GL)	Happy Valley Reservoir (GL)	Myponga Reservoir (GL)
July	5.5	2.4	0.3
August	6.5	2.4	0.5
September	9.8	2.4	0.7
October	12.1	3.2	1.2
November	12.7	4.5	1.5
December	13.0	4.7	1.8
January	11.3	4.7	1.9
February	9.0	4.7	1.6
March	5.0	4.7	1.2
April	3.9	3.3	0.8
May	1.4	2.4	0.3
June	2.8	1.9	0.3

Table B.6: Southern System - '6 Weeks Demand' Target Storage Component

Month	Mount Bold Reservoir (GL)	Happy Valley Reservoir (GL)	Myponga Reservoir (GL)
July	2.1	2.4	0.4
August	2.1	2.4	0.4
September	3.7	2.4	0.7
October	5.2	3.2	1.0
November	4.8	4.5	1.2
December	5.9	4.7	1.3
January	4.6	4.7	1.1
February	4.1	4.7	0.9
March	1.5	4.7	0.6
April	1.2	3.3	0.4
May	1.1	2.4	0.3
June	2.5	1.9	0.3

Table B.7: Southern System - '4 Weeks Demand' Target Storage Component

Month	Mount Bold Reservoir (GL)	Happy Valley Reservoir (GL)	Myponga Reservoir (GL)
July	1.4	0.8	0.2
August	1.8	0.4	0.2
September	2.6	0.4	0.4
October	2.8	1.2	0.5
November	2.2	2.5	0.6
December	2.6	2.7	0.7
January	1.9	2.7	0.6
February	1.7	2.7	0.5
March	1.4	1.7	1.3
April	1.4	0.8	0.2
May	1.4	0.4	0.2
June	1.4	0.8	0.1

Table B.8: Southern System - '2 Weeks Demand' Target Storage Component

B.2.5 Pipeline Pump Cost Curve Coefficients

The cost curve coefficients for pumping via the Murray Bridge-Onkaparinga pipeline are given below in Tables B.9 and B.10

Month	Pumping Volume Node 1 (GL)	Pumping Volume Node 2 (GL)	Pumping Volume Node 3 (GL)	Pumping Volume Node 4 (GL)	Pumping Volume Node 5 (GL)
July	2.4251	4.3894	7.8058	11.3326	14.7859
August	2.4251	4.3894	7.8058	11.3326	14.7859
September	2.5482	4.6174	5.7363	11.6557	15.3281
October	2.5482	4.6174	5.7363	11.6557	15.3281
November	2.7863	5.5407	11.2584	14.8055	14.8055
December	2.7863	5.5407	11.2584	14.8055	14.8055
January	2.7863	5.5407	11.2584	14.8055	14.8055
February	2.7863	5.5407	11.2584	14.8055	14.8055
March	2.5482	4.6174	5.7363	11.6557	15.3281
April	2.5482	4.6174	5.7363	11.6557	15.3281
May	2.5482	4.6174	5.7363	11.6557	15.3281
June	2.4251	4.3894	7.8058	11.3326	14.7859

Table B.9: Murray Bridge-Onkaparinga Monthly Pump Cost Curve Volume Coefficients

Month	Pumping Cost Node 1 ($M)	Pumping Cost Node 2 ($M)	Pumping Cost Node 3 ($M)	Pumping Cost Node 4 ($M)	Pumping Cost Node 5 ($M)
July	0.13555	0.26574	0.51141	0.78454	1.13489
August	0.13555	0.26574	0.51141	0.78454	1.13489
September	0.14221	0.27880	0.35733	0.78703	1.13850
October	0.14221	0.27880	0.35733	0.78703	1.13850
November	0.15490	0.33230	0.76520	1.10691	1.10691
December	0.15490	0.33230	0.76520	1.10691	1.10691
January	0.15490	0.33230	0.76520	1.10691	1.10691
February	0.15490	0.33230	0.76520	1.10691	1.10691
March	0.14221	0.27880	0.35733	0.78703	1.13850
April	0.14221	0.27880	0.35733	0.78703	1.13850
May	0.14221	0.27880	0.35733	0.78703	1.13850
June	0.13555	0.26574	0.51141	0.78454	1.13489

Table B.10: Murray Bridge-Onkaparinga Monthly Pump Cost Curve Cost Coefficients

B.2.6 Reservoir Evaporation Coefficients

Evaporation coefficients used within the model for Mount Bold, Happy Valley and Myponga Reservoirs are given in Table B.11 below.

Month	Mount Bold Reservoir		Happy Valley Reservoir		Myponga Reservoir	
	A	B	A	B	A	B
July	0.02668	0.00258	0.03133	0.00442	0.00161	0.00469
August	0.03770	0.00364	0.04427	0.00624	0.00228	0.00663
September	0.04524	0.00437	0.05312	0.00749	0.00273	0.00796
October	0.06670	0.00644	0.07832	0.01104	0.00403	0.01173
November	0.08062	0.00778	0.09466	0.01334	0.00487	0.01418
December	0.10324	0.00997	0.12122	0.01709	0.00623	0.01816
January	0.11484	0.01109	0.13484	0.01901	0.00693	0.02020
February	0.09976	0.00963	0.11713	0.01651	0.00602	0.01754
March	0.08236	0.00795	0.09670	0.01363	0.00497	0.01448
April	0.05742	0.00554	0.06742	0.00950	0.00347	0.01010
May	0.03248	0.00314	0.03814	0.00538	0.00196	0.00571
June	0.02668	0.00258	0.03133	0.00442	0.00161	0.00469

Table B.11: Southern System Reservoir Evaporation Coefficients

B.2.7 Southern System Demand Zone Transfer Costs and Capacities

For all months in the year the adopted monthly transfer capacity from the Happy Valley to the Myponga demand zone is 3,000 ML. The corresponding monthly transfer capacity from the Myponga to the Happy Valley demand zone is 300 ML. The transfer between demand zones utilises the head difference between the reservoirs and the supply zones and the adopted cost associated with this transfer is $0.01 per ML. Facility is available within the HOMA model to vary these monthly transfer capacities and costs as modifications are made to the southern supply system.

B.2.8 Southern System Water Filtration Plant Costs and Capacities

For all months in the year the adopted monthly capacity for the Happy Valley Water Filtration Plant is 25,000 ML. The corresponding monthly capacity for the Myponga Water Filtration Plant is 1,500 ML. The operating costs assumed for both water filtration plants in the southern system for each month of the year is $40 per ML. Facility is available within the HOMA model to vary these monthly capacities and costs as modifications are made to the southern supply system.

B.3 Northern System

B.3.1 Reservoir Storage Capacities

The reservoir storage capacities used in the model optimisation run have been taken from the following historical measured reservoir storage capacities shown in Table B.12 below.

Reservoir	Storage (GL)
Warren	4.770
South Para	44.800
Barossa	4.510
Little Para	20.800
Millbrook	16.500
Kangaroo Creek	19.000
Hope Valley	3.470

Table B.12: Northern System Reservoir Storage Capacities

B.3.2 Initial Reservoir Storage Levels

The starting reservoir storage levels for a model optimisation run have been taken from the following start of year historical reservoir storage levels shown in Table B.13 below. When a run is carried out over multiple years, the model uses only the initial year historical storages and calculates the remaining start of year storages from the previous years run.

Reservoir	Storage (GL)
Warren	4.770
South Para	44.800
Barossa	4.510
Little Para	20.147
Millbrook	16.500
Kangaroo Creek	15.280
Hope Valley	3.470

Table B.13: Northern System - 'Start of Run' Reservoir Storage Levels

B.3.3 Target Storage Levels

The target storage levels used in the operation of the northern metropolitan system comprise two components :

- Minimum Operating Level component
- Demand Storage Component

The minimum operating level components for all months in the water year for the Warren, South Para, Barossa, Little Para, Millbrook Kangaroo Creek and Hope Valley Reservoirs are shown in Table B.14 below.

Reservoir	Storage (GL)
Warren	1.0
South Para	11.1
Barossa	3.5
Little Para	5.0
Millbrook	3.0
Kangaroo Creek	0.5
Hope Valley	1.2

Table B.14: Northern System - Nominal Minimum Operating Levels (for all months)

The demand storage components corresponding to the '8 weeks demand', '6 weeks demand', '4 weeks demand' and '2 weeks demand' criteria, described in Section 5.2.2.2 for the northern metropolitan system are presented in Tables B.15, B.16, B.17, B.18, B.19, B.20, B.21 and B.22.

Month	Warren Reservoir (GL)	South Para Reservoir (GL)	Barossa Reservoir (GL)	Little Para Reservoir (GL)
July	0.6	2.4	0.7	0.1
August	0.9	2.6	0.7	0.7
September	1.5	2.5	0.8	2.0
October	2.0	2.7	0.9	3.1
November	2.5	3.5	0.8	3.6
December	2.6	3.2	1.0	3.6
January	2.3	2.8	1.0	3.2
February	1.8	2.0	1.0	2.5
March	1.2	1.7	0.9	1.3
April	0.8	1.7	0.6	0.5
May	0.3	0.1	0.7	0.1
June	0.2	1.0	0.4	0.1

Table B.15: Northern System - '8 Weeks Demand' Demand Storage Component - South Para and Little Para Subsystems

Month	Millbrook Reservoir (GL)	Kangaroo Creek Reservoir (GL)	Hope Valley Reservoir (GL)
July	3.9	1.9	0.6
August	4.8	3.6	0.6
September	7.4	6.0	0.7
October	9.5	6.7	1.4
November	10.9	7.0	1.9
December	11.4	6.7	2.1
January	10.4	5.8	2.3
February	8.4	4.8	2.1
March	5.8	3.0	2.0
April	4.4	2.2	1.5
May	2.1	0.6	1.0
June	1.9	0.3	0.8

Table B.16: Northern System - '8 Weeks Demand' Demand Storage Component - Torrens Subsystem

Month	Warren Reservoir (GL)	South Para Reservoir (GL)	Barossa Reservoir (GL)	Little Para Reservoir (GL)
July	0.4	1.4	0.7	0.1
August	0.6	1.6	0.7	0.4
September	0.9	1.6	0.8	1.2
October	1.3	1.5	0.9	2.1
November	1.7	2.1	0.8	2.5
December	1.9	2.1	1.0	2.6
January	1.6	1.7	1.0	2.2
February	1.3	1.3	1.0	2.0
March	1.0	0.8	0.9	1.1
April	0.6	1.1	0.6	0.4
May	0.3	0.1	0.7	0.1
June	0.2	1.0	0.4	0.1

Table B.17: Northern System - '6 Weeks Demand' Target Storage Component - South Para and Little Para Subsystems

Month	Millbrook Reservoir (GL)	Kangaroo Creek Reservoir (GL)	Hope Valley Reservoir (GL)
July	2.8	1.0	0.6
August	3.0	2.0	0.6
September	4.7	3.7	0.7
October	6.5	4.2	1.4
November	7.4	4.1	1.9
December	8.1	4.2	2.1
January	7.4	3.5	2.3
February	6.3	3.1	2.1
March	4.4	1.8	2.0
April	3.2	1.2	1.5
May	2.1	0.6	1.0
June	1.9	0.3	0.8

Table B.18: Northern System - '6 Weeks Demand' Target Storage Component - Torrens Subsystem

Month	Warren Reservoir (GL)	South Para Reservoir (GL)	Barossa Reservoir (GL)	Little Para Reservoir (GL)
July	0.2	0.6	0.7	0.1
August	0.3	0.8	0.7	0.1
September	0.5	0.8	0.8	0.6
October	0.8	0.6	0.9	1.3
November	1.0	1.0	0.8	1.6
December	1.3	1.2	1.0	1.8
January	1.2	0.8	1.0	1.5
February	0.9	0.7	1.0	1.5
March	0.7	0.2	0.9	0.9
April	0.4	0.7	0.6	0.3
May	0.3	0.1	0.7	0.1
June	0.2	0.9	0.4	0.1

Table B.19: Northern System - '4 Weeks Demand' Target Storage Component - South Para and Little Para Subsystems

Month	Millbrook Reservoir (GL)	Kangaroo Creek Reservoir (GL)	Hope Valley Reservoir (GL)
July	1.9	0.4	0.6
August	1.8	0.7	0.6
September	2.7	1.9	0.7
October	4.2	2.2	1.4
November	4.6	2.1	1.9
December	5.5	2.2	2.1
January	5.1	1.6	2.3
February	4.6	1.6	2.1
March	3.3	0.7	2.0
April	2.2	0.4	1.5
May	1.9	0.6	1.0
June	1.8	0.2	0.8

Table B.20: Northern System - '4 Weeks Demand' Target Storage Component - Torrens Subsystem

Month	Warren Reservoir (GL)	South Para Reservoir (GL)	Barossa Reservoir (GL)	Little Para Reservoir (GL)
July	0.1	0.0	0.7	0.0
August	0.2	0.1	0.7	0.1
September	0.3	0.0	0.8	0.3
October	0.4	0.0	0.8	0.6
November	0.5	0.1	0.8	0.8
December	0.6	0.1	1.0	0.9
January	0.6	0.0	0.9	0.7
February	0.5	0.0	0.9	0.7
March	0.4	0.0	0.5	0.5
April	0.2	0.1	0.6	0.2
May	0.1	0.0	0.4	0.0
June	0.1	0.2	0.4	0.0

Table B.21: Northern System - '2 Weeks Demand' Target Storage Component - South Para and Little Para Subsystems

Month	Millbrook Reservoir (GL)	Kangaroo Creek Reservoir (GL)	Hope Valley Reservoir (GL)
July	0.9	0.0	0.6
August	0.9	0.0	0.6
September	1.3	0.7	0.7
October	2.1	0.4	1.4
November	2.3	0.1	1.9
December	2.8	0.0	2.1
January	2.6	0.0	1.9
February	2.3	0.0	1.9
March	1.6	0.0	1.4
April	1.1	0.0	1.0
May	1.0	0.0	0.7
June	0.9	0.0	0.5

Table B.22: Northern System - '2 Weeks Demand' Target Storage Component - Torrens Subsystem

B.3.4 Pipeline Pump Cost Curve Coefficients

Pump cost curve coefficients for each of the northern system pump stations, Millbrook, Swan Reach-Stockwell and Mannum-Adelaide are given below in Tables B.23, B.24, B.25 and B.26.

Pumped Volume (GL)	Pumping Cost ($M)
0.000	0.00000
3.750	0.09250
7.500	0.18500

Table B.23: Millbrook Pump Station Monthly Pump Cost Curve Coefficients

Month	Pumping Volume Node 1 (GL)	Pumping Cost Node 1 ($M)	Pumping Volume Node 2 (GL)	Pumping Cost Node 2 ($M)
July	0.9399	0.09868	1.9670	0.23860
August	0.9399	0.09868	1.9670	0.23860
September	0.7885	0.07871	2.0454	0.23925
October	0.7885	0.07871	2.0454	0.23925
November	0.9573	0.09461	1.9756	0.23320
December	0.9573	0.09461	1.9756	0.23320
January	0.9573	0.09461	1.9756	0.23320
February	0.9573	0.09461	1.9756	0.23320
March	0.7885	0.07871	2.0454	0.23925
April	0.7885	0.07871	2.0454	0.23925
May	0.7885	0.07871	2.0454	0.23925
June	0.9399	0.09868	1.9670	0.23860

Table B.24: Swan Reach-Stockwell Monthly Pump Cost Curve Coefficients

Month	Pumping Volume Node 1 (GL)	Pumping Volume Node 2 (GL)	Pumping Volume Node 3 (GL)	Pumping Volume Node 4 (GL)	Pumping Volume Node 5 (GL)
July	2.1203	3.8097	5.6120	8.0040	10.3651
August	2.1203	3.8097	5.6120	8.0040	10.3651
September	1.1871	2.2305	3.8996	8.0388	10.4102
October	1.1871	2.2305	3.8996	8.0388	10.4102
November	1.2981	2.6778	3.7666	7.7648	10.0553
December	1.2981	2.6778	3.7666	7.7648	10.0553
January	1.2981	2.6778	3.7666	7.7648	10.0553
February	1.2981	2.6778	3.7666	7.7648	10.0553
March	1.1871	2.2305	3.8996	8.0388	10.4102
April	1.1871	2.2305	3.8996	8.0388	10.4102
May	1.1871	2.2305	3.8996	8.0388	10.4102
June	2.1203	3.8097	5.6120	8.0040	10.3651

Table B.25: Mannum-Adelaide Monthly Pump Cost Curve Volume Coefficients

Month	Pumping Cost Node 1 ($M)	Pumping Cost Node 2 ($M)	Pumping Cost Node 3 ($M)	Pumping Cost Node 4 ($M)	Pumping Cost Node 5 ($M)
July	0.14478	0.28130	0.42744	0.62766	0.85485
August	0.14478	0.28130	0.42744	0.62766	0.85485
September	0.07605	0.15190	0.28588	0.62966	0.85757
October	0.07605	0.15190	0.28588	0.62966	0.85757
November	0.08283	0.18105	0.26586	0.61219	0.83378
December	0.08283	0.18105	0.26586	0.61219	0.83378
January	0.08283	0.18105	0.26586	0.61219	0.83378
February	0.08283	0.18105	0.26586	0.61219	0.83378
March	0.07605	0.15190	0.28588	0.62966	0.85757
April	0.07605	0.15190	0.28588	0.62966	0.85757
May	0.07605	0.15190	0.28588	0.62966	0.85757
June	0.14478	0.28130	0.42744	0.62766	0.85485

Table B.26: Mannum-Adelaide Monthly Pump Cost Curve Cost Coefficients

B.3.5 Reservoir Evaporation Coefficients

The evaporation coefficients used within the model for Warren, South Para, Barossa, Little Para, Millbrook, Kangaroo Creek and Hope Valley Reservoirs are given below in Tables B.27 B.28 and B.29.

Month	Warren Reservoir		South Para Reservoir		Barossa Reservoir	
	A	B	A	B	A	B
July	0.00475	0.00676	0.02166	0.00266	0.00312	0.00464
August	0.00600	0.00854	0.02736	0.00336	0.00394	0.00586
September	0.00863	0.01228	0.03933	0.00483	0.00566	0.00842
October	0.01400	0.01994	0.06384	0.00784	0.00918	0.01366
November	0.01725	0.02456	0.07866	0.00966	0.01132	0.01684
December	0.02150	0.03062	0.09804	0.01204	0.01410	0.02098
January	0.02413	0.03435	0.11001	0.01351	0.01583	0.02355
February	0.02050	0.02919	0.09348	0.01148	0.01345	0.02000
March	0.01713	0.02439	0.07809	0.00959	0.01123	0.01671
April	0.01125	0.01602	0.05130	0.00630	0.00738	0.01098
May	0.00675	0.00961	0.03078	0.00378	0.00443	0.00659
June	0.00475	0.00676	0.02166	0.00266	0.00312	0.00464

Table B.27: South Para Subsystem Reservoir Evaporation Coefficients

Month	Little Para Reservoir	
	A	B
July	0.00965	0.00228
August	0.01219	0.00288
September	0.01753	0.00414
October	0.02845	0.00672
November	0.03505	0.00828
December	0.04369	0.01032
January	0.04902	0.01158
February	0.04166	0.00984
March	0.03480	0.00822
April	0.02286	0.00540
May	0.01372	0.00324
June	0.00965	0.00228

Table B.28: Little Para Subsystem Reservoir Evaporation Coefficients

Month	Millbrook Reservoir		Kangaroo Creek Reservoir		Hope Valley Reservoir	
	A	B	A	B	A	B
July	0.01480	0.00348	0.00828	0.00180	0.00536	0.00548
August	0.01813	0.00426	0.01014	0.00221	0.00657	0.00671
September	0.02590	0.00609	0.01449	0.00315	0.00938	0.00959
October	0.04477	0.01053	0.02505	0.00545	0.01621	0.01658
November	0.05550	0.01305	0.03105	0.00675	0.02010	0.02055
December	0.07030	0.01653	0.03933	0.00855	0.02546	0.02603
January	0.07585	0.01784	0.04244	0.00923	0.02747	0.02809
February	0.06401	0.01505	0.03581	0.00779	0.02318	0.02370
March	0.05217	0.01227	0.02919	0.00635	0.01889	0.01932
April	0.03700	0.00870	0.02070	0.00450	0.01340	0.01370
May	0.02109	0.00496	0.01180	0.00257	0.00764	0.00781
June	0.01406	0.00331	0.00787	0.00171	0.00509	0.00521

Table B.29: Torrens Subsystem Reservoir Evaporation Coefficients

B.3.6 Northern System Demand Zone Transfer Costs and Capacities

For all months in the year the adopted monthly transfer capacity from the Barossa to the Little Para demand zone is 960 ML. The corresponding monthly transfer capacity from the Little Para to the Barossa demand zone is also 960 ML. The transfer from the Barossa to the Little Para demand zone utilises the head difference between the reservoirs and the supply zones and the adopted cost associated with this transfer is $0.01 per ML. Transfer from Little Para to Barossa demand zone is currently not possible due to head differences. A cost of $10,000 per ML has been set as this transfer cost, representing tankering costs between the demand zones.

For all months in the year the adopted monthly transfer capacity from the Little Para to the Anstey Hill demand zone is 960 ML. The corresponding monthly transfer capacity from the Anstey Hill to the Little Para demand zone is also 1,860 ML.. Transfer from Little Para to Anstey Hill demand zone is currently not possible due to head differences. A cost of \$10,000 per ML has been set as this transfer cost, representing tankering costs between the demand zones. The transfer from the Anstey Hill to the Little Para demand zone utilises the head difference between the reservoirs and the supply zones and the adopted cost associated with this transfer is \$0.01 per ML.

For all months in the year the adopted monthly transfer capacity from the Anstey Hill to the Hope Valley demand zone is 1,860 ML. The corresponding monthly transfer capacity from the Hope Valley to the Anstey Hill demand zone is also 1,440 ML. The transfer from the Anstey Hill to the Hope Valley demand zone utilises the head difference between the reservoirs and the supply zones and the adopted cost associated with this transfer is \$0.01 per ML. Transfer from Hope Valley to Anstey Hill demand zone is currently not possible due to head differences. A cost of \$10,000 per ML has been set as this transfer cost, representing tankering costs between the demand zones.

Facility is available within the HOMA model to vary these monthly transfer capacities and costs as modifications are made to the northern supply system.

B.3.7 Northern System Water Filtration Plant Costs and Capacities

For all months in the year the adopted monthly capacity for the Barossa, Little Para, Anstey Hill and Hope Valley Water Filtration Plants is 4,800 ML , 4,800 ML , 9,300 ML and 7,200 ML respectively. The operating costs assumed for each of the water filtration plants in the northern system for each month of the year is \$40 per ML. Facility is available within the HOMA model to vary these monthly capacities and costs as modifications are made to the northern supply system.

Appendix C

The 'Walking Party' Participants

This appendix outlines the skills and expertise of the personnel who participated in the 'walking party' process used to identify the critical components of the Adelaide bulk water transfer system and assess their reliability attributes. The personnel were :

Lionel Rodrigues	-	Operations and Maintenance Engineer, Mechanical and Electrical, Murray Bridge, EWS

Lionel Rodrigues is intimately involved with the operation and maintenance of the mechanical and electrical components of the major pumping systems. His experience and knowledge of the current conditions within these systems, made his participation in the 'walking party' process vital.

Hanley Pullar	-	Operations Superintendent, Murray Bridge, EWS

Hanley's primary role is to oversee the day to day operation of the Murray Bridge-Onkaparinga and Mannum-Adelaide pumping systems. He is also directly involved in the monitoring, maintenance and repair of the key components used within these systems. Over 30 years of experience within the EWS enabled Hanley to provide valuable 'hands on' input during the 'walking party' process.

John Minney - Manager, Headworks,
Headworks and Treatment, EWS

John Minney is involved in the management and operation of the headworks system. His knowledge and familiarity of the system makes him an ideal candidate to provide input to the 'walking party' process. John has had extensive experience with the construction, repair, maintenance and operation of the Adelaide system.

Dave Kerry - Supervising Engineer, Pipe Systems,
Engineering Services, EWS

Dave Kerry has been involved in the design and construction of pipework components within the EWS and has over 20 years of expertise in this area. His expertise in this area provided valuable input during the 'walking party' process.

Tony Soar - Supervising Engineer, Electrical,
Engineering Services, EWS

Tony Soar's expertise is in the area of high voltage power supply within the EWS. He has over 25 years of experience in this area and has a thorough knowledge of the design, operation and maintenance requirements for these components within the major pumping system.

Bob Jordan - Project Manager,
F.R. Mayfield Pty. Ltd.

Bob Jordan has been involved with major mechanical and electrical construction projects with F.R. Mayfield for over 30 years. He has been directly involved with construction contracts for the metropolitan Adelaide bulk water transfer system. His firm is often called upon to provide services to the EWS involving mechanical and electrical components. Bob's wealth of experience with pumps, pump motors and switchboards associated with the Adelaide system makes him an ideal candidate to participate in the 'walking party' process.

Andrew Jessup - Operations Engineer,
Operations Support Branch, EWS

Andrew Jessup's role within the EWS involves the planning and monitoring of the month by month and longer term operation of the headworks system. His familiarity with the 'bigger picture' associated with the operation of the system, makes Andrew an ideal candidate to participate in the 'walking party' process.

Bill Hagger - Substation Maintenance Supervisor
Marlston, ETSA

Bill Hagger is involved with the maintenance and repair of ETSA's high voltage electricity distribution system. Bill plays a role as 'trouble shooter' in the event of the failure of key components within this system. He has a breadth of knowledge regarding the maintenance and repair requirements for high voltage electrical systems and provided a wealth of experience applicable to the power supply systems for the major pumping systems.

Phil. Crawley - Research Engineer,
Operations Support Branch, EWS

Phil. Crawley was the facilitator and coordinator of the 'walking party' process.

Appendix D

Metropolitan Adelaide Bulk Water Transfer System Reliability

D.1 Introduction

This appendix describes the metropolitan Adelaide bulk water transfer system and the components identified as critical to the operation of this system. Assessed reliability estimates obtained during the 'walking party' process are detailed for these critical components.

D.2 Summary of Reliability Information obtained for the Metropolitan Adelaide Bulk Water Transfer System

The three major pumping systems supplying the metropolitan Adelaide water supply system have been described in Section 4.2. Each of these systems comprises three pump stations in series that progressively lift water from the River Murray over the Mount Lofty Ranges and into reservoirs in the headworks system or directly to a water filtration plant.

Reliability information for each of these pumping systems are detailed below. Where information is common across a number of pump stations or pipelines, detailed information will be provided in the first description of this type of component and subsequent instances will be referenced to this initial description.

D.2.1 Murray Bridge-Onkaparinga Pumping System

Components at the Murray Bridge-Onkaparinga no. 1 pump station were considered to be typical of the components at the other two pump stations in the Murray Bridge-Onkaparinga pumping system. Any differences between the three pump stations have been discussed.

The Murray Bridge-Onkaparinga pumping system has been described in Section 4.2.1 of Chapter 4 and a longitudinal schematic section of the pipeline presented in Figure 4.3.

A schematic detailing the components identified as critical to the operation of the pipeline at the Murray Bridge-Onkaparinga no. 1 pump station is shown in Figure D.1. The critical power supply components common to each of the three pump stations in the pumping system are shown in Figure D.2. Murray Bridge-Onkaparinga no. 2 and no. 3 pump stations have similar reliability attributes and a schematic detailing the critical components in these pump stations is shown in Figure D.3. Details concerning these components are discussed below.

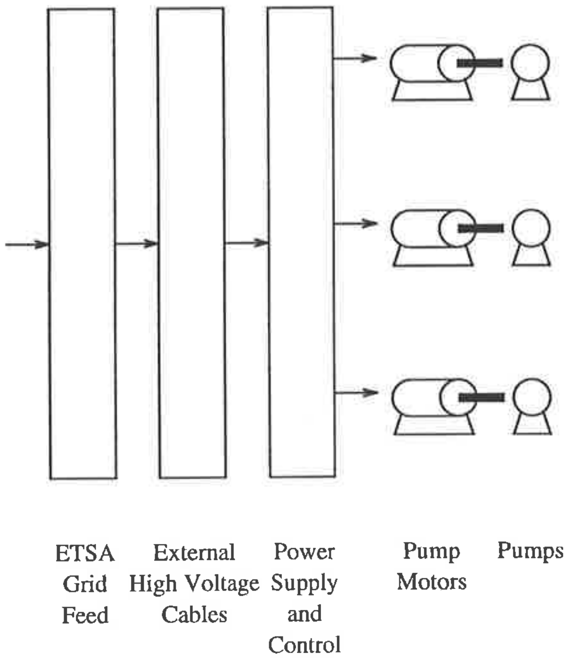


Figure D.1: Murray Bridge-Onkaparinga Pump Station No. 1 Critical Component Schematic

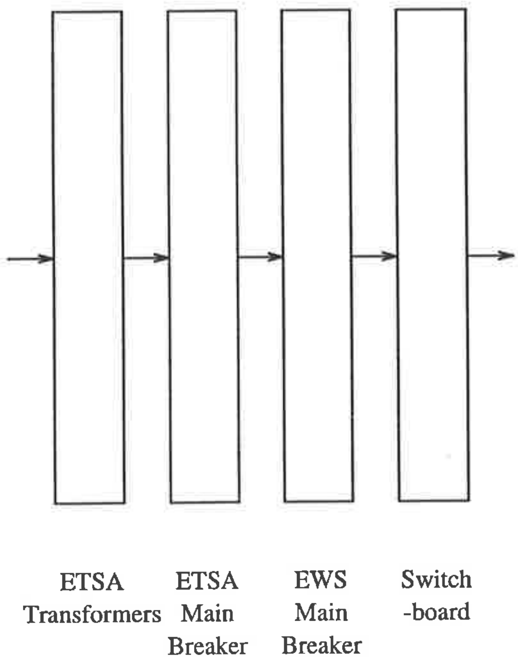


Figure D.2: Murray Bridge-Onkaparinga Pump Stations Critical Power Supply and Control Component Schematic

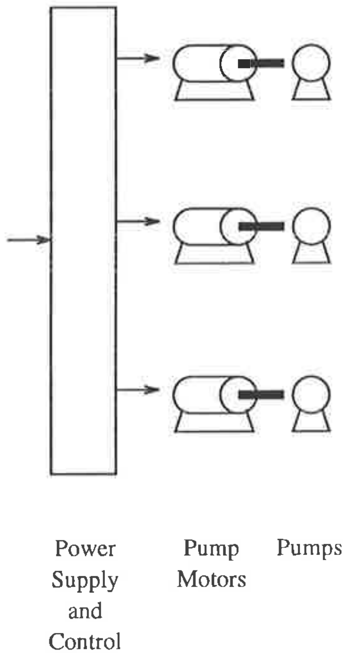


Figure D.3: Murray Bridge-Onkaparinga Pump Station No. 2 and No. 3 Critical Component Schematic

D.2.1.1 ETSA Grid feed to the Murray Bridge-Onkaparinga no. 1 Pump Station

The power supply to the Murray Bridge-Onkaparinga no. 1 pump station is shown schematically in Figure 4.25.

The Murray Bridge-Onkaparinga no. 1 pump station is supplied with power from the ETSA grid through a mesh bus at the Mobilong substation. Mobilong substation itself has two main circuit breakers, both of which would need to fail to shut down the substation. These circuit breakers could be replaced in a week. Mobilong substation can be fed from the three alternate sources of Mannum, Mount Barker and Tailem Bend. A single line feeds directly from this substation to the ETSA switchyard at the pump station. At the no. 2 and 3 pump stations it is possible to feed the ETSA switchyard at these locations from two directions. The mean repair time for a failure on the line from Mobilong substation to the switchyard at Murray Bridge-Onkaparinga no. 1

pump station is estimated as one week. Since pump stations 2 and 3 have a dual feed facility, an ETSA grid feed failure could be isolated and the pump station made operational within a day. During the 'walking party' process it was concluded that only the reliability of the Murray Bridge-Onkaparinga no. 1 pump station ETSA grid feed needed to be considered. An estimate was made that the failure of the single line feed to this pump station would have a frequency of one in one thousand years and a mean repair time of one week.

D.2.1.2 ETSA Transformers on the Murray Bridge-Onkaparinga Pumping System

There are two ETSA transformers at Murray Bridge-Onkaparinga no. 1 pump station that are used to provide the 11 kV supply. The details of these transformers are :

132/11 kV (12.5 MVA)

These transformers are identical to those at pump stations 2 and 3.
Table D.1 gives the rated power drawn by the pumps at each of the Murray Bridge-Onkaparinga pump stations together with the transformer capacities available at each pump station.

Location	Total Pump Power Rating (kW)	Transformer Capacity required (kW)	Number of Pump Sets available using only one transformer
Pump Station No. 1	8997	10600	3
Pump Station No. 2	18120	21400	1
Pump Station No. 3	17650	20800	1

Table D.1: Murray Bridge-Onkaparinga Pipeline Transformer Capacities

These transformers are visually inspected by ETSA personnel every three months. A more detailed inspection of each transformer is undertaken every four to five years. These more detailed inspections involve the sampling of the oil coolant used in the transformer. A chemical analysis of the oil sample is undertaken and, if necessary, the oil within the transformer is replaced. Trace elements or gas detected in the oil also indicate potential problems with the transformer insulating elements.

Around South Australia, ETSA has four to five hundred large high voltage transformers in service. These transformers can be categorised into two types : those having manual tap changers and those having automatic tap changers. Of the two types of transformers, those having automatic tap changers are characterised by a much higher level of reliability. Each of the transformers used to supply the pump stations on the three major pumping systems have automatic tap changers.

The incidence of failure of any of the four to five hundred transformers is approximately one in four to five years. This therefore gives a failure probability of an individual transformer of the order of one in sixteen hundred to two thousand five hundred years. These estimates are probably conservative since the EWS transformers have automatic tap changers and the sample set includes transformers that have manual tap changers.

In the event of the failure of one of these high voltage transformers, spare transformers are held at Marleston. The size of the transformers are such that a crane and special trucking requirements are necessary for their relocation. It is estimated that to get a replacement transformer back in service would take a maximum total time of one week.

During the 'walking party' process, it was concluded that in the event of a single transformer failure at any of the three pump stations, for safety reasons the pumping system would need to be completely taken out of service for a mean time of one week. For the purpose of the study, the failure frequency should be adopted as one in sixteen hundred years.

D.2.1.3 Main Circuit Breaker to the ETSA transformers at the Murray Bridge-Onkaparinga Pump Stations

The main ETSA circuit breaker (132 kV) is an essential component of the power supply system to each of the pump stations. As shown in Figure 4.25, this circuit breaker is located within the ETSA switchyard and provides protection to the ETSA grid in the event of an electrical fault occurring within the ETSA switchyard. It can also be used by EWS to isolate the pumping system in the event of an earth fault occurring within its 11 kV main switch. Regular maintenance of these circuit breakers is carried out by ETSA. A minor overhaul is carried out every four and a half years and a major overhaul is carried out every nine years. A spare system circuit breaker is always available in the event of the failure of a particular circuit breaker.

During the 'walking party' process, it was estimated that the mean repair time for these circuit breakers is one week and that the frequency of failure is one in five hundred years. These estimates were based upon ETSA's experience with these type of components in use throughout the electricity supply network in South Australia.

D.2.1.4 EWS Main Circuit Breakers adjacent to the ETSA transformers in the Murray Bridge-Onkaparinga Pumping System

The EWS main circuit breaker provides isolation to the EWS 11 kV system and protection in the event of overload, short circuit or earth leakage. Separate pump overload protection is provided by individual circuit breakers for each pump set. On the Murray Bridge-Onkaparinga pumping system, the three EWS circuit breakers are of the 'minimum oil' rather than 'vacuum' type. When a 'minimum oil' circuit breaker operates on load, the arc causes burning of the oil and because of the low oil volume, the level of contaminants in the oil rises relatively quickly. A check of these circuit breakers is undertaken every fifty operations. It is intended to replace these 'minimum oil' with 'vacuum' circuit breakers on the Murray Bridge-Onkaparinga pumping system in the near future, when funds become available. These 'minimum oil' circuit breakers are recognised as having a lower level of reliability than 'vacuum' circuit breakers.

In the event of a failure of this main circuit breaker, this component could be bypassed using a portable ETSA 11 kV circuit breaker. This circuit breaker would be connected in-line as a temporary solution until a replacement distribution board was designed and constructed.

During the 'walking party' process, it was estimated that the mean time to install a portable circuit breaker and bring the system safely back into operation would be one week. The frequency of failure of the 'minimum oil' circuit breakers was estimated as one in fifty years.

D.2.1.5 High Voltage Cables from the EWS Power Distribution Substation to the Pump Station Switchboard

The Murray Bridge-Onkaparinga no. 1 pump station is located on the banks of the River Murray while the ETSA switchyard supplying power to the pump station is located 400 metres away, adjacent to the system control centre. Two, 0.3 square inch (195 mm^2) three core aluminium XLPE double tape armoured 11 kV cables are used to transmit power from the EWS main circuit breaker adjacent to the ETSA switchyard, to the pump station high voltage switchboard.

Following the assessment day, Bill Hagger (ETSA) reviewed the available cable resources that are held in stock within ETSA. If these two cables were to fail, then although ETSA does not have identical replacement cable, they would be able to provide three 300 mm^2 single core aluminium XLPE cables in parallel to meet the load requirement. Cable of this size is regularly required by ETSA and consequently, significant lengths are maintained in stock. Replacement of these cables would also involve termination of the cables at each end. Although of large size, the necessary resources required to terminate these cables are also

available within ETSA.
It was estimated during the 'walking party' process that the mean time to replace these cables and restore power to the pumps, given the availability of spare cable, would be one week. The failure frequency for these cables was estimated as one in fifty years.

D.2.1.6 High Voltage Switchboard at the Murray Bridge-Onkaparinga Pump Stations

Each of the pump stations on the Murray Bridge-Onkaparinga and Mannum-Adelaide pumping systems have a high voltage switchboard. These switchboards provide an interface between the ETSA transformers and the pump motors.
ETSA has approximately two hundred similar high voltage switchboards in service throughout their power supply network in South Australia. During the last thirty years, there has been one failure event involving these two hundred switchboards. This single failure event was caused by the installation of an unsuitable circuit breaker in the switchboard. The failure did not require complete replacement of the switchboard and the fault was repaired within a few days. Although these high voltage switchboards will be of different designs according to the specific site conditions in which they are installed, they do indicate that the failure probability for switchboards of this type are of the order of one in six thousand years or greater.
In the event of a partial switchboard failure, the rearrangement of the power supply to use the undamaged portion of the switchboard should be possible in two to three days. In the event of a complete switchboard failure, the pumps could be made available using portable 11 kV circuit breakers. These circuit breakers would be installed within the pump station and cabling provided to bypass the switchboard. During the 'walking party' process it was estimated that the mean time to bring the system back into operation, using portable circuit breakers was one week. This solution, although temporary, would suffice

until a replacement switchboard could be designed and constructed. A failure frequency of one in six thousand years was also estimated.

D.2.1.7 Pump Motors on the Murray Bridge-Onkaparinga Pumping System

There are three pump sets at the Murray Bridge-Onkaparinga no. 1 pump station. Each of these pump sets comprise a pump and a pump motor. At the Murray Bridge-Onkaparinga no. 2 pump station there are three tandem pump sets. Each of the tandem pump sets comprise a primary pump and pump motor and a secondary pump and pump motor. At the no. 3 pump station there are three single stage pump sets comprising a pump and pump motor. All pumps and pump motors in the Murray Bridge-Onkaparinga pumping system were commissioned in 1973. Table D.2 gives details of the pump motors installed in the Murray Bridge-Onkaparinga pumping system.

Pump Station	Pump Unit	Manufacturer	Power/Motor (kW)	No.
No. 1 Pump Station	Primary	Hitachi	2830	3
No. 2 Pump Station	Primary	Mitsubishi	1380	3
	Secondary	Mitsubishi	4660	3
No. 3 Pump Station	Primary	Mitsubishi	5670	3

Table D.2: Murray Bridge-Onkaparinga Pumping System Pump Motor Details

A pump set, used to supply water to the Murray Bridge township, is also located in the Murray Bridge pump station. This Hitachi motor is rated at 507 kW. The additional capacity of this pump set, above the Murray Bridge township water demands, is small and has not been considered in relation to the metropolitan Adelaide bulk water transfer system. Over the twenty years of operation of the Murray Bridge-Onkaparinga pumping system, there have been no major failures of the pump motors.

In the event of a failure requiring a total rewind for an individual motor, there are few companies within Australia that would have both the expertise and the facilities to properly rewind motors of these sizes. One company that does have such expertise and equipment is Bob Whites Electrix in Geelong. In the event of a complete motor rewind being necessary, the quickest solution would be to send the motor to Geelong to have it rewound. Following discussions with Jeff White of Bob Whites Electrix it was estimated that a complete rewind of one of the motors on the Murray Bridge-Onkaparinga pumping system under normal conditions would take of the order of eight weeks. In the event that 'round the clock' work was undertaken this figure could be reduced to five weeks. During the 'walking party' process it was estimated that the failure frequency of an individual pump motor failure requiring a complete motor rewind is one in fifty years.

D.2.1.8 Pumps on the Murray Bridge-Onkaparinga Pumping System

On-going maintenance is carried out on the individual pumps of the Murray Bridge-Onkaparinga pumping system. Table D.3 gives the details associated with each of the pumps on the Murray Bridge-Onkaparinga pumping system.

Pump Station	Pump Unit	Manufacturer	Pump Details	No.
No. 1 Pump Station	Primary	Kelly and Lewis	1200 V.S M.F.D.	3
No. 2 Pump Station	Primary	Kelly and Lewis	825/900 SDS-DV	3
	Secondary	Kelly and Lewis	675/825 SDS-DV	3
No. 3 Pump Station	Primary	Kelly and Lewis	675/875 SDS-DV	3

Table D.3: Murray Bridge-Onkaparinga Pumping System Pump Details

These pumps have performed well over their twenty year life to date. The potential problems that could arise with these pumps are :

- pump shafts breaking
- impeller corroding
- sealing rings wearing

In each of these situations, the necessary resources are available within EWS to repair the pumps. As a minimum, three days are required to remove the pumps for repair and a further three days for their reinstatement.

Individual pumps in the Murray Bridge pumping system were installed in 1970 and are subject to regular inspection and maintenance. During this regular inspection and maintenance, identified potential problems are rectified as necessary. An unscheduled failure of a pump would result in reduced capacity of the pumping system. In the event of such a failure, if the pumping capacity required were less than the remaining available capacity, the system would remain unaffected by the pump failure. The pump could therefore be repaired in a scheduled manner. In the event of a pump failure occurring when all pumps were required, there would be an urgency for the pump to be repaired as quickly as possible.

During the 'walking party' process it was estimated that under normal conditions the mean repair time of an individual pump would be three weeks. If 'round the clock' repair work were undertaken, this time could be reduced to two weeks. A consensus reached during the 'walking party' process for an individual pump failure frequency on the Murray Bridge-Onkaparinga pumping system was one in fifty years.

D.2.2 Mannum-Adelaide Pumping System

Components at the Mannum-Adelaide no. 1 pump station are typical of the components at the other two pump stations in the Mannum-Adelaide pumping system. A longitudinal pipeline schematic for the Mannum-Adelaide pumping system has been previously presented in Figure 4.12 in Section 4.2.2 of Chapter 4.

A schematic detailing the components identified as critical to the operation of the pumping system at the Mannum-Adelaide no. 1 pump station is shown in Figure D.4. Details of the critical power supply components common to each of the three pump stations in the pumping system are presented in Figure D.5. Mannum-Adelaide no. 2 and 3 pump stations have similar reliability attributes and a schematic detailing the critical components in these pump stations is shown in Figure D.6. Two types of pumps and pump motors have been shown in these schematics. Type 'A' pumps are vertical centrifugal pumps while type 'B' pumps are horizontal pumps. Details regarding the critical components shown in these schematics are discussed below.

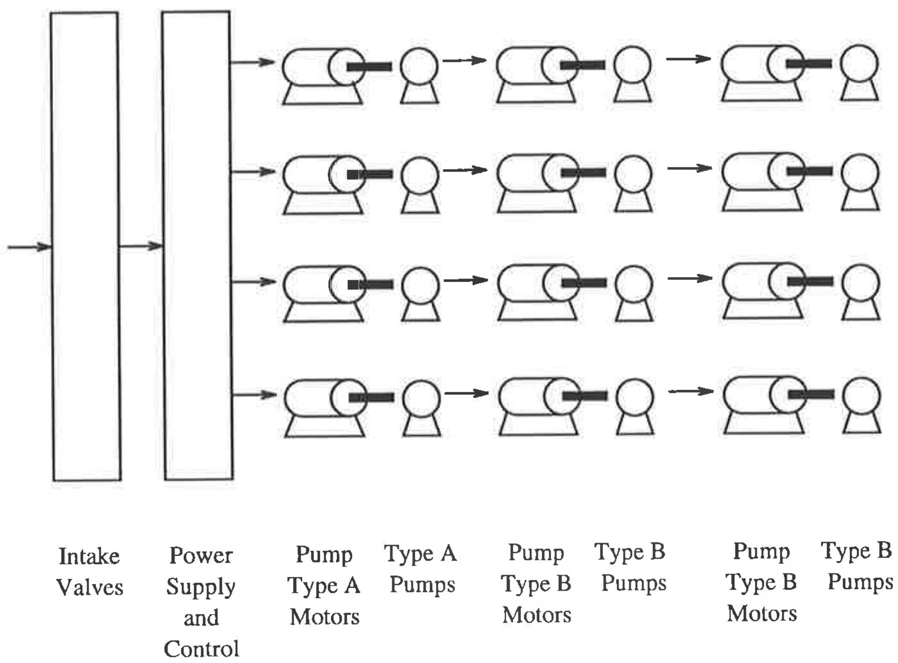


Figure D.4: Mannum-Adelaide Pump Station No. 1 Critical Component Schematic

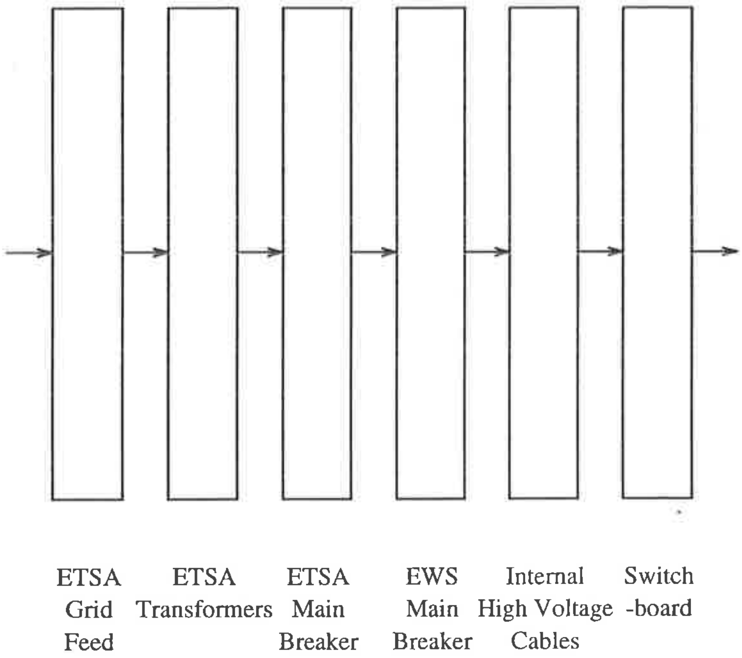


Figure D.5: Mannum-Adelaide Pump Stations Critical Power Supply and Control Component Schematic

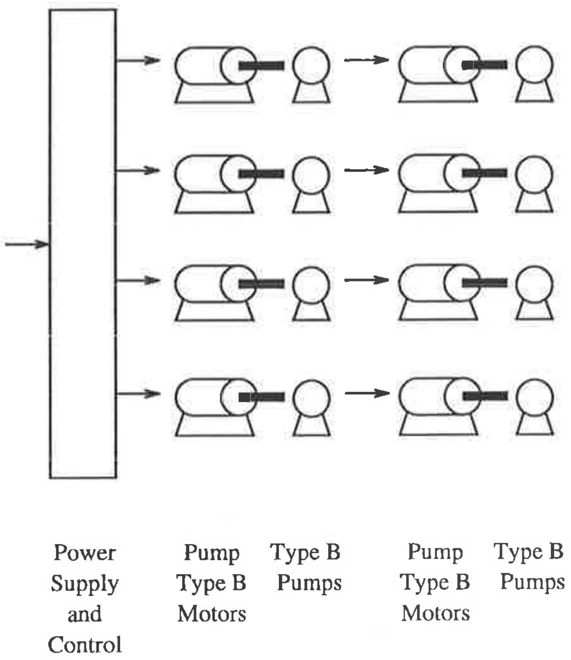


Figure D.6: Mannum-Adelaide Pump Station No. 2 and No. 3 Critical Component Schematic

D.2.2.1 ETSA Grid feed to the Mannum-Adelaide No. 1 Pump Station

The power supply to the Mannum-Adelaide no. 1 pump station is shown schematically in Figure D.7.

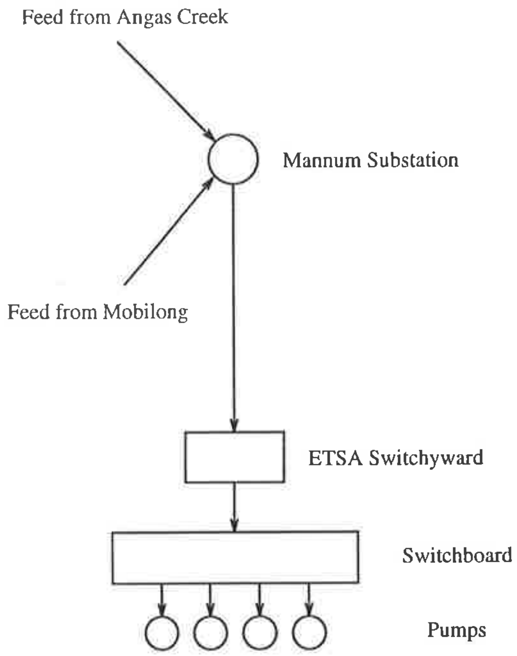


Figure D.7: Mannum-Adelaide Pump Station No. 1 Power Supply Schematic

The Mannum-Adelaide no. 1 pump station is supplied with power from the ETSA grid through a mesh bus at the Mannum substation. Mannum substation has two main circuit breakers, both of which would need to fail for the substation to shut down. These circuit breakers could be replaced in a week. Mannum substation can be fed from the two alternate sources of Angus Creek and Mobilong. A single line feeds direct from this substation to the ETSA switchyard at the pump station. During the 'walking party' process, it was estimated that the mean repair time for a failure of the power supply line from the Mannum substation to the switchyard at Mannum-Adelaide no. 1 pump station was 1 week, with a failure frequency of one in one thousand years.
During the 'walking party' process, it was concluded that the mean repair times

and failure frequencies for the nos. 2 and 3 pump stations in the Mannum-Adelaide pumping system and the three Swan Reach-Stockwell pump stations could be assumed the same as the Mannum-Adelaide no. 1 pump station.

D.2.2.2 ETSA Transformers at the Mannum-Adelaide Pump Stations

There are dual ETSA transformers at the Mannum-Adelaide no. 1 pump station that are used to provide a 3.3 kV power supply to the pump station. These transformers are 132 to 3.3 kV step-down transformers rated at 7.5 MVA. The transformers at pump stations no. 2 and 3 are similar to those at the no. 1 pump station being 132 to 3.3 kV step-down transformers rated at 6.0 MVA.

Table D.4 gives the rated power drawn by the pumps at each of the Mannum-Adelaide pump stations, the transformer capacity requirements for the pump station to be fully operational and the number of pump sets that could be operated if only one of the two available transformers were operational.

Location	Total Pump Power Rating (kW)	Transformer Capacity required (kW)	Number of Pump Sets available using only one transformer
Pump Station No. 1	12028	14200	2
Pump Station No. 2	8652	10200	2
Pump Station No. 3	9552	11300	2

Table D.4: Mannum-Adelaide Pumping System Transformer Details

Maintenance is undertaken by ETSA on these transformers in the same manner as the transformers at the Murray Bridge-Onkaparinga and Mannum- Adelaide pump stations. These transformers have automatic tap changers. The likelihood of failure of one of these transformer units is therefore similar to those

at the Murray Bridge-Onkaparinga and Mannum-Adelaide pump stations.
In the event of the failure of one of these high voltage transformers, spare transformers are held at Marleston. The size of the transformers are such that a crane and special trucking requirements are necessary for their relocation.
During the 'walking party' process, it was concluded that in the event of a single transformer failure at any of the three pump stations, for safety reasons, the pumping system would need to be completely taken out of service for a mean time of one week and for the purpose of the study, the failure frequency should be taken as one in sixteen hundred years (the same failure frequency as the transformers on the Murray Bridge-Onkaparinga pumping system).

D.2.2.3 Main Circuit Breaker to the ETSA transformers at the Mannum-Adelaide Pump Stations

The main circuit breaker (132 kV) provides protection to the ETSA grid in the event of a fault occurring within the ETSA switchyard. Regular maintenance of these circuit breakers is carried out by ETSA. This maintenance is consistent for all circuit breakers in the ETSA grid and has already been described in the section on the 'Main Circuit Breaker to the ETSA transformers at the Murray Bridge-Onkaparinga Pump Stations'.
During the 'walking party' process it was concluded that the mean repair time for the main ETSA circuit breaker was one week and a failure frequency of one in five hundred years should be adopted for the study.

D.2.2.4 EWS Main Circuit Breakers adjacent to the ETSA transformers in the Mannum-Adelaide Pumping System

The EWS main circuit breakers provide isolation and protection to the pumps in the event of overload, short circuit or earth leakage protection in the 3.3 kV power supply system. Separate pump overload protection is also provided at each pump set by individual pump motor circuit breakers. On the Mannum-Adelaide pumping system, the EWS circuit breakers are all of the 'vacuum' type.

In the event of a failure of one of these main circuit breakers, it would be possible to bypass this component using a portable ETSA 11 kV circuit breaker. This portable circuit breaker could be connected in-line as a temporary solution until a replacement distribution board was designed and constructed. In response to questioning, participants of the 'walking party' affirmed that an 11 kV circuit breaker would function adequately in the 3.3 kV system.

During the 'walking party' process, it was estimated that the mean time to install a portable circuit breaker and bring the system safely back into operation would be one week. The frequency of failure of the vacuum circuit breakers was estimated as one in five hundred years.

D.2.2.5 High Voltage Cables internal to the Mannum-Adelaide Pump Stations

An issue raised during the 'walking party' process at the Mannum-Adelaide no. 1 pump station was the location and nature of the internal high voltage cables in the Mannum-Adelaide pump stations. These cables are laid in common cable ways between the switchboard and the individual pumps. In the event of the failure of one of these cables, adjacent cables may also be damaged. In the Mannum-Adelaide pump stations, the majority of the cables between the switchboard and the pump motors are 3.3 kV, three core, paper-lead. This type of cable is considered more susceptible to damage than the more modern XLPE cables.

In the event of a failure of one or more of these cables, they could be replaced with appropriately bundled 300 mm^2 single core aluminium XLPE PVC cables. This would provide both the load capacity and fault tolerance requirements. This cabling in most cases is of short length only (less than 20 metres).

As previously noted on page 564 under the heading 'High Voltage Cables from the EWS Power Distribution Substation to the Pump Station Switchboard' for the Murray Bridge-Onkaparinga pumping system, cables of this size and length are maintained in store by ETSA together with the necessary resources for cable termination. In the event of a major failure of the internal cabling in one of the pump stations, resources from ETSA would be made available to expedite the repair process.

It was estimated during the 'walking party' process, that the mean time to provide temporary arrangements to restore power to the pumps in the event of internal cable failure within a pump station would be one week. Because of the age and nature of these cables, the failure frequency was estimated as one in twenty years.

D.2.2.6 High Voltage Switchboards in the Mannum-Adelaide Pump Stations

Each of the pump stations on the Mannum-Adelaide pumping system utilise a high voltage switchboard. These switchboards provide an interface between the ETSA transformers and the pump motors. The switchboards at each of these pump stations are of the same design and constructed of the same make and capacity components. The Millbrook pump station switchboard, when originally designed and constructed, also utilised these same components. In recent years, the Millbrook switchboard has been replaced. The undamaged components of this switchboard are now available as spares for the switchboards at each of the three Mannum-Adelaide pump stations. In the event of a single component failing on any of these three switchboards, spare components are available and the switchboard could be made operational in a few days.

During the 'walking party' process, the reliability of these switchboards was considered similar to the switchboards at each of the Murray Bridge-Onkaparinga pump stations. In the event of a complete switchboard failure, a temporary bypass system would need to be installed while a new switchboard was manufactured. This temporary bypass system could be constructed utilising the portable 11 kV circuit breakers that ETSA uses in emergency situations together with appropriate high voltage cabling. It was noted that the pump stations in the Mannum-Adelaide pumping system use 3.3 kV rather than 11 kV and that the circuit breakers would be in excess of the requirements.

It was estimated during the 'walking party' process that the mean time to provide temporary arrangements to restore power to the pumps, in the event of a major switchboard failure within one of the Mannum-Adelaide pump stations, would be one week. The failure frequency was estimated as one in two thousand years (the same failure frequency as the switchboards in the Murray Bridge-Onkaparinga pumping system).

D.2.2.7 Intake Valves at Mannum-Adelaide No. 1 Pump Station

There are individual intake valves linking the River Murray to each of the type 'A' pump units in the Mannum-Adelaide no. 1 pump station. These valves are used to isolate the pump station from the River Murray. During the 'walking party' process, a concern raised by participants during the visit to the Mannum-Adelaide no. 1 pump station was regarding the consequence of the body of one of these valves failing. If this occurred, the primary pump well would be flooded, drowning the type 'A' pump unit. The pump motors are located above the level to which the water would rise in the event of a valve failure and so would remain unaffected. In order to re-establish pumping at the Mannum-Adelaide no. 1 pump station, it would be necessary to stoplog off the inlet channel to the pump station, pump the area dry, remove the valve and flange off the pipe.

It was estimated during the 'walking party' process that the mean repair time for these valves would be two weeks under normal repair conditions. If the repair was undertaken in a 'crisis' situation and 'round the clock' resources were made available, it was estimated that the mean repair time could be reduced to one week. The failure frequency for these intake valves was estimated as one in twenty years. The possibility of failure of these valves had not been raised during the individual interview process, but arose in the 'walking party' process during the visit to the Mannum-Adelaide no. 1 pump station.

These intake valves are specific to the no. 1 pump station on the Mannum-Adelaide pumping system.

D.2.2.8 Pump Motors at the Mannum-Adelaide Pump Stations

There are four pump sets at each of the pump stations in the Mannum-Adelaide pumping system. Each of these pump sets comprise a pump and a pump motor. The majority of pump motors in use in the Mannum-Adelaide pumping system are designed and constructed by Mitsubishi. Details of these pump motors are given in Table D.5.

Location	Motor Maker	Pump Type	Pump Unit	Power/Motor (kW)	Total No
No. 1 Pump Station	Mitsubishi	'A'	Primary	470	4
	Mitsubishi	'B'	Primary and Secondary	1194 x 2 (2388)	6
	Bruce Peebles	'B'	Primary and Secondary	1492 x 2 (2984)	2
No. 2 Pump Station	Mitsubishi	'B'	Primary and Secondary	1044 x 2 (2088)	6
	Mitsubishi	'B'	Primary and Secondary	1194 x 2 (2388)	2
No. 3 Pump Station	Mitsubishi	'B'	Primary and Secondary	1194 x 2 (2388)	8

Table D.5: Mannum-Adelaide Pumping System Pump Motor Details

In the event of a pump motor failure requiring the total rewind of the motor, details given for the pump motors on the Murray Bridge-Onkaparinga pumping system would also be applicable to the Mannum-Adelaide pumping system. Following discussions with Jeff White of Bob Whites Electrix in Geelong, it is estimated that a complete rewind of one of the motors on the Mannum-Adelaide pumping system under normal conditions would take of the order of three weeks. In the event that 'round the clock' work was undertaken this figure could be reduced to two weeks. The failure frequency, estimated during the 'walking party' process for these pump motors was one in fifty years (the same failure frequency as the pump motors in the Murray Bridge-Onkaparinga pumping system).

D.2.2.9 Pumps at the Mannum-Adelaide Pump Stations

Table D.6 gives details of the pumps at each of the pump stations in the Mannum-Adelaide pumping system.

Location	Pump Type	Pump Unit	Manufacturer	Pump Details	No
No. 1 Pump Station	'A'	Primary	Thompson	760 mm. Vertical	4
	'B'	Primary and Secondary	Thompson	760/850 SS Class C	6
	'B'	Primary and Secondary	Kelly and Lewis	530/600 SDS-DV	2
No. 2 Pump Station	'B'	Primary and Secondary	Thompson	760/825 SS Class C	6
	'B'	Primary and Secondary	Thompson	760/845 SS Class C	2
No. 3 Pump Station	'B'	Primary and Secondary	Thompson	760/845 SS Class C	8

Table D.6: Mannum-Adelaide Pumping System Pump Details

Ongoing maintenance is carried out on the individual pumps of the Mannum-Adelaide pumping system. During this regular inspection and maintenance, potential problems identified are rectified as required. These pumps have performed well over their forty year life to date. Problems that could arise with these pumps include : a pump shaft breaking, an impeller corroding and the sealing rings requiring replacement. In each of these situations, the necessary equipment to repair these components would be available. During the 'walking party' process it was estimated that under normal conditions repair of an individual pump would require up to two weeks. If 'round the clock' repair work were undertaken, the mean repair time for an individual pump was estimated

to be one week. Because of the age and condition of these pumps, the failure frequency for an individual pump was estimated as one in twenty years.
There is some flexibility in the Mannum-Adelaide pumping system for the relocation of the individual Thompson pumps and Mitsubishi pump motors between pump stations in the event of multiple failures at a single pump station. Although these relocated pumps may not operate at peak efficiency, it would still be possible for pumping to be carried out. In this way, if two pumps or pump motors were to fail at a particular pump station, the capacity of the pumping system would only be reduced to three quarters of full capacity, as a pump or pump motor from a different pump station could be relocated to the station where two had failed. This flexibility also exists for the pumps and pump motors at the no. 2 and no. 3 pump stations in the Swan Reach pumping system.

D.2.3 Millbrook Pump Station

A schematic detailing the components identified as critical to the operation of the Millbrook pump station is shown in Figure D.8.
Details concerning each of these components are discussed below.

D.2.3.1 ETSA Grid feed to the Millbrook Pump Station

The pump station at Millbrook was commissioned in 1971 and is supplied with power from the ETSA grid from the Millbrook substation. Millbrook substation itself has one hydraulically operated main circuit breaker (132 kV). In the event of a failure this circuit breaker could be replaced in a week. Millbrook substation is fed via a 'T' off the Angas Creek-Northfield 132 kV line.
It was concluded during the 'walking party' process that the reliability attributes for the ETSA grid feed to the Millbrook pump station would be the same as those for the ETSA grid feed to the Mannum-Adelaide pumping system, that is, a mean repair time of one week and a failure frequency of one in

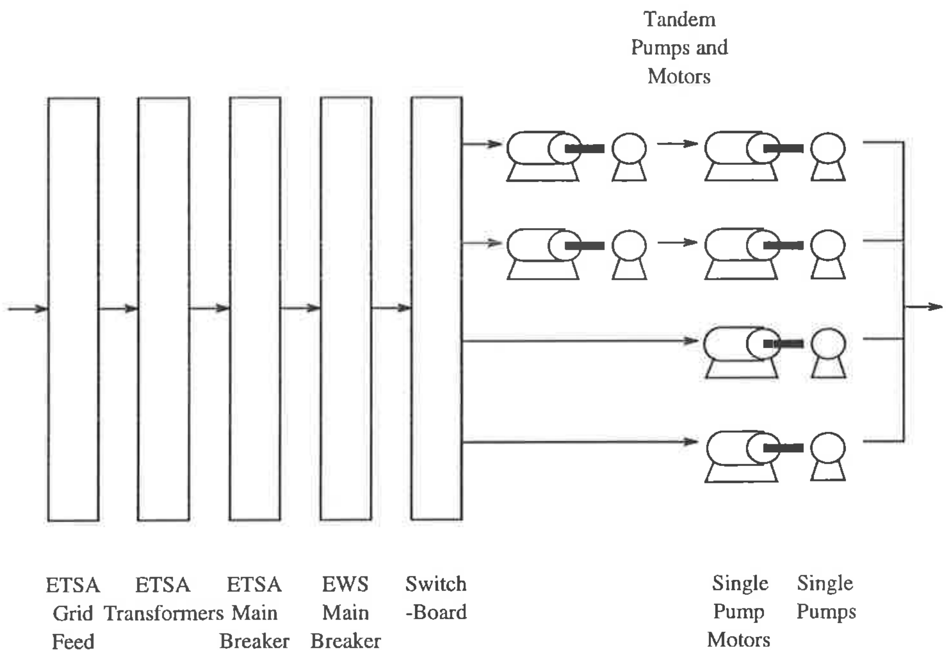


Figure D.8: Millbrook Pump Station Critical Component Schematic

one thousand years.

D.2.3.2 ETSA Transformers at Millbrook Pump Station

There are two ETSA transformers at the Millbrook pump station that are used to provide 3.3 kV supply to the pump station. The details of these transformers are :

132/3.3 kV (5 MVA)

These transformers are similar to those at Mannum-Adelaide pump station nos. 1, 2 and 3.

Table D.7 gives the rated power drawn by the pumps at the Millbrook pump station.

Each of the two transformers has sufficient capacity to meet the power requirements of two tandem units within the pump station (eg. there is two

Location	Total Pump Power Rating (kW)	Transformer Capacity required (kW)	Number of Pump Sets available using only one transformer
Millbrook Pump Station	6000	7100	2 Tandem Units

Table D.7: Millbrook Pump Station Transformer Details

thirds standby capacity at the pump station). Regular maintenance is carried out by ETSA on these transformers. Details of this maintenance program are the same as that undertaken for the transformers associated with the Murray Bridge-Onkaparinga and Mannum-Adelaide pumping systems. In the event of the failure of one of these high voltage transformers, spare transformers are held at Marleston. Transportation of these high voltage transformers would require the same special arrangements as previously described for the Murray Bridge-Onkaparinga and Mannum-Adelaide pumping systems.
During the 'walking party' process, it was concluded that a mean repair time of one week and a failure frequency of one in sixteen hundred years were applicable to these transformer components.

D.2.3.3 Main Circuit Breaker to the ETSA transformers at Millbrook Pump Station

The main circuit breaker (132 kV) provides protection to the ETSA grid in the event of a fault in the ETSA switchyard. Regular maintenance is carried out at this switchyard in the same manner as that previously described for the Murray Bridge-Onkaparinga pump stations. The circuit breaker at Millbrook pump station is of a slightly different type to those at the Murray Bridge-Onkaparinga and Mannum-Adelaide pump stations. Spares for this circuit breaker are available and it is estimated that the pump station could be brought back into full operation in one week. The reliability of circuit breakers of the type in use at the Millbrook pump station is known to be less than other ETSA circuit breakers in use elsewhere.

During the 'walking party' process, estimates of one week for the mean repair time and one in fifty years for the failure frequency were adopted for this circuit breaker.

D.2.3.4 EWS Main Circuit Breakers adjacent to the ETSA transformers at the Millbrook Pump Station

In the event of a failure of the main circuit breaker, it would be possible to bypass this component of the pump station using a portable ETSA 11 kV circuit breaker. This circuit breaker could be connected in-line until a replacement distribution board was designed and constructed. In response to questioning, participants of the 'walking party' affirmed that an 11 kV circuit breaker would function adequately in a 3.3 kV system.

During the 'walking party' process, it was estimated that the mean time to install a portable circuit breaker and bring the system safely back into operation would be one week. The frequency of failure of the 'vacuum' type main circuit breaker in use at the Millbrook pumping station was estimated as one in five hundred years.

D.2.3.5 High Voltage Switchboard at Millbrook Pump Station

The high voltage switchboard at Millbrook pump station has recently been replaced. At the time of replacement, a spare main circuit breaker and spare motor breaker were also purchased and is available in the event of either of these components failing.

The reliability of this switchboard is similar, if not better than those in the Murray Bridge pumping system. In the event of a partial switchboard failure, the rearrangement of the power supply to use the undamaged portion of the switchboard should be possible in a few days. In the event of a complete switchboard failure, the pumps could be made operational using portable 11 kV circuit breakers. These circuit breakers would be installed within the pump station and cabling provided to bypass the switchboard. During the 'walking party' process it was estimated that the mean time to bring the system back into operation using portable circuit breakers was one week. This solution would be satisfactory until a replacement switchboard could be designed and constructed. A failure frequency of one in six thousand years was estimated for a complete switchboard failure at the Millbrook pump station.

D.2.3.6 Pump Motors at the Millbrook Pump Station

There are four pump sets at the Millbrook pump station, two of these are tandem sets with the other two being single sets. The tandem pump sets comprise two pumps and two pump motors while the single sets comprise one pump and one pump motor. Details of the pump and pump motor configuration is presented in Figure D.8.

At the Millbrook pump station six new pump motors were installed in 1984. These pump motors are manufactured by Mitsubishi and are rated at 1000 kW and have a reputation as being reliable motors. Over the period of operation of the Millbrook pump station, there have been no major problems with the pump motors.

In the event of a failure requiring the total rewind of a pump motor, the description given for the pump motors in the Mannum-Adelaide pump stations is applicable to the pump motors in the Millbrook pump station.

There is ongoing work being undertaken examining the possibility of upgrading both the pumps and the pump motors in the Millbrook Pump station. This would result in an improvement in the reliability of these components.

The consensus reached during the 'walking party' approach was that the mean repair time for the pump motors at the Millbrook pump station is two weeks and the failure frequency is one in fifty years.

D.2.3.7 Pumps at the Millbrook Pump Station

Details of the pumps at Millbrook pump station are given in the Table D.8.

Location	Pump Unit	Manufacturer	Pump Set Details	No.
Millbrook Pump Station	Primary and Secondary	Thompson	750/750 Class C 1 Stage	2 x 2
	Primary	Kelly and Lewis	400/400 SCO 2 Stage	2

Table D.8: Millbrook Pump Station Pump Details

Of the four pump sets at the Millbrook pump station, the tandem pumps were initially installed in the Mannum-Adelaide pumping system and later relocated to the Millbrook pump station.

Ongoing maintenance is carried out on the individual pumps in the Millbrook pump station. During this regular inspection and maintenance, identified potential problems are rectified as necessary. These pumps have performed well over their forty year life to date. Problems that could arise with these pumps have been described in the previous section for the Mannum-Adelaide pump station pumps. During the 'walking party' process it was estimated that under normal conditions repair of an individual pump would require up to two weeks. In the event of a 'crisis' situation where 'round the clock' repair work was undertaken, the mean repair time for an individual pump was estimated to be one week. Because of the age and condition of these pumps, the failure frequency for an individual pump was estimated as one in twenty years.

D.2.4 Swan Reach-Stockwell Pumping System

During the 'walking party' process, the Swan Reach-Stockwell pumping system was not directly considered because of time limitations and the smaller impact of this pumping system on the overall reliability of the metropolitan Adelaide headworks system.

The age and form of construction of components on the Swan Reach-Stockwell pumping system are similar to both the Mannum-Adelaide and Murray Bridge-Onkaparinga pumping systems. Component reliability information obtained during the 'walking party' process for these two pumping systems have been assumed for the components on the Swan Reach-Stockwell pumping system.
The ETSA grid feed, ETSA transformers, ETSA main circuit breakers, pumps and pump motors associated with the Swan Reach-Stockwell pumping system are similar to those on the Mannum-Adelaide pumping system and so these reliability data have been assumed.
The EWS main circuit breakers and switchboards associated with the Swan Reach-Stockwell pumping system are similar to those on the Murray Bridge-Onkaparinga pumping system and so these reliability data have been assumed.

D.2.5 The Mannum-Adelaide, Swan Reach-Stockwell and Murray Bridge-Onkaparinga Pipelines

The Mannum-Adelaide, Swan Reach-Stockwell, and Murray Bridge-Onkaparinga pipelines were commissioned in 1954, 1969 and 1973 respectively. All three pipelines were constructed using spun, concrete lined, steel pipes.
Severe damage to each of these pipelines could be result from a 'direct hit' by a truck. A 'glancing blow', where the pipeline was struck obliquely would result in only minor damage and would put the pipeline out of service for one to two days. There are no locations on any of the three pipelines where a major road is running at right angles to a section of pipeline with the associated potential for a 'direct hit'.
Providing ongoing inspection and maintenance is carried out on the pipelines, then problems arising such as localised corrosion, resulting in small holes in the pipeline, could be patched with a rolled piece of plate and repaired in a day. If it was required to replace a short section of pipe, it was estimated during the 'walking party' process that the mean time to repair the pipe was three days.

In the event of the failure of one of the major line valves it would be possible to replace the line valve with a 'cotton reel'. It was estimated that a repair of this type could be achieved in less than a week. The major purpose of these line valves is to enable the isolation of sections of the pipelines in the event of a leak. Isolation of a section of the pipeline would limit the loss of water to that section of the pipeline.

It was concluded during the 'walking party' process that failure of a section of pipe linking the pump stations on the three pumping systems could be repaired in less than a week and therefore need not be considered as critical to the bulk water transfer system reliability.

D.3 Summary of Bulk Water Transfer System Critical Component Reliability Results

Tables D.9, D.10, D.11 and D.12 summarise the results obtained for the three major pumping systems together with the Millbrook pump station, used to supply metropolitan Adelaide.

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
MBO No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	External High Voltage Cables	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233
MBO No. 2 Pump Station	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233
MBO No. 3 Pump Station	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	35	1 in 50	0.998082
	Pumps	14	1 in 50	0.999233

Table D.9: Murray Bridge-Onkaparinga (MBO) Pumping System Critical Component Reliability Data

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Man-Ad. No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082
	Intake Valves	7	1 in 20	0.999041
Man-Ad. No. 2 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082
Man-Ad. No. 3 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Internal High Voltage Cables	7	1 in 20	0.999041
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082

Table D.10: Mannum-Adelaide Pumping System Critical Component Reliability Data

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Millbrook Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 50	0.999616
	EWS Main Circuit Breaker	7	1 in 500	0.999962
	Switchboard	7	1 in 6000	0.999997
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 20	0.998082

Table D.11: Millbrook Pump Station Critical Component Reliability Data

Location	Component Description	Mean Repair Time (Days)	Failure Frequency (Years)	Daily Availability
Swan Reach Stockwell No. 1 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233
Swan Reach Stockwell No. 2 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233
Swan Reach Stockwell No. 3 Pump Station	ETSA Grid Feed	7	1 in 1000	0.999981
	ETSA Transformers	7	1 in 1600	0.999988
	ETSA Main Circuit Breaker	7	1 in 500	0.999962
	EWS Main Circuit Breaker	7	1 in 50	0.999616
	Switchboard	7	1 in 2000	0.999990
	Pump Motors	14	1 in 50	0.999233
	Pumps	14	1 in 50	0.999233

Table D.12: Swan Reach-Stockwell Pumping System Critical Component Reliability Data

Appendix E

Comparison of Historical and Generated Inflow and Rainfall Data

The multisite multiperiod autoregressive data data generation developed by Baker and Dandy and described in Section 4.4 has been used to generate 10,000 years of synthetic data for the Adelaide hills catchments.

Details of the statistics of the generated data and historical data are compared in this appendix. Discussion of this comparison is given in Section 4.4.2.

Using the techniques described in Section 4.4.3, a flow frequency analysis of the generated and historical inflow data has also been undertaken. The purpose of this analysis is to compare the statistical properties of the generated streamflow with the historical streamflow records for a range of cumulative flow periods. Discussion of the results presented in this appendix is made in Section 4.4.3.3.

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-278.	-266.	-447.	-160.	-19.	-44.
Maximum	979.	999.	462.	1244.	11111.	20407.
Mean	67.7	118.6	40.8	216.8	1110.4	3548.7
Median	2.5	10.5	23.5	159.5	365.5	678.0
std. dev.	239.4	288.6	158.3	271.3	2012.6	5841.4
Skew	1.946	1.538	0.014	1.563	3.307	1.745
% zero	45	40	45	14	5	2
10% conf.	451.0	535.0	212.0	605.0	3437.0	13512.0
90% conf.	-116.0	-75.0	-119.0	-25.0	32.0	71.0

Table E.1: South Para Historical Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	75.	136.	-37.	15.	-202.	-216.
Maximum	29532.	29312.	27248.	18694.	5991.	901.
Mean	6024.0	7937.6	5633.9	2912.2	716.2	142.9
Median	4096.5	4987.5	3476.5	1621.5	258.0	70.0
std. dev.	7130.0	8233.8	5876.8	3905.0	1136.6	238.9
Skew	1.893	1.137	1.386	2.086	2.761	1.339
% zero	0	0	2	0	10	26
10% conf.	13299.0	18881.0	12574.0	8838.0	1715.0	494.0
90% conf.	348.0	514.0	347.0	144.0	-27.0	-83.0

Table E.2: South Para Historical Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-262.	-248.	-352.	-149.	-19.	-38.
Maximum	915.	1344.	439.	1375.	19978.	90611.
Mean	69.1	120.4	43.0	220.1	1220.2	4380.9
Median	24.2	51.1	42.8	158.1	417.0	955.8
std. dev.	220.5	284.2	158.3	270.3	2672.1	11814.0
Skew	1.246	1.693	-0.015	1.622	4.331	4.821
% zero	45	41	39	18	3	5

Table E.3: South Para Generated Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	-148.	-307.	-335.	-7.	-179.	-200.
Maximum	70955.	86788.	62005.	59135.	6771.	1085.
Mean	6589.1	8837.6	6268.2	3556.9	717.3	144.1
Median	3406.8	4757.7	3359.8	1292.0	362.9	91.8
std. dev.	10219.5	12961.2	9241.6	7707.5	1104.4	238.0
Skew	3.471	3.382	3.363	4.758	2.847	1.386
% zero	2	3	4	1	17	30

Table E.4: South Para Generated Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	0.	1.	27.	59.	195.	205.
Maximum	2138.	3419.	607.	1425.	4527.	12169.
Mean	300.6	304.9	255.2	404.7	997.8	2594.1
Median	264.0	232.0	261.5	369.0	625.0	1262.0
std. dev.	331.3	506.2	144.5	243.0	1045.0	2803.1
Skew	4.168	5.532	0.441	1.805	2.271	1.887
% zero	0	0	0	0	0	0
10% conf.	409.0	356.0	409.0	687.0	2152.0	5867.0
90% conf.	52.0	92.0	79.0	146.0	295.0	435.0

Table E.5: Myponga Historical Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	360.	462.	290.	119.	119.	0.
Maximum	14638.	15509.	14122.	6032.	1702.	594.
Mean	4391.4	4480.7	3054.5	1478.6	580.1	320.3
Median	3168.5	4124.5	2010.0	898.0	541.0	297.0
std. dev.	3685.0	3213.1	3082.3	1395.7	352.8	165.0
Skew	1.196	0.969	1.827	1.795	1.351	-0.051
% zero	0	0	0	0	0	0
10% conf.	10130.0	8288.0	7075.0	3788.0	844.0	532.0
90% conf.	682.0	831.0	568.0	290.0	211.0	87.0

Table E.6: Myponga Historical Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	3.	18.	-15.	48.	196.	203.
Maximum	1422.	1639.	725.	1282.	8057.	26593.
Mean	295.1	285.3	256.2	405.1	1029.1	2795.7
Median	231.7	208.3	237.1	360.8	655.7	1546.3
std. dev.	245.8	263.8	146.4	231.0	1163.9	3914.2
Skew	1.897	2.453	0.716	1.220	3.272	3.361
% zero	1	0	1	0	0	0

Table E.7: Myponga Generated Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	212.	-270.	269.	132.	94.	-95.
Maximum	27786.	18519.	25053.	9915.	2115.	724.
Mean	4542.7	4554.3	3182.4	1532.2	587.0	321.1
Median	3283.2	3808.8	1978.5	1053.0	500.2	320.1
std. dev.	4441.8	3445.8	3775.7	1586.1	364.7	163.7
Skew	2.547	1.466	3.109	2.725	1.575	-0.032
% zero	0	2	0	0	0	3

Table E.8: Myponga Generated Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-461.	-727.	-856.	-460.	-40.	299.
Maximum	1895.	3735.	762.	4201.	11793.	33107.
Mean	314.5	255.7	75.6	496.4	1864.4	5150.9
Median	258.0	166.0	111.0	345.5	986.0	1542.5
std. dev.	471.4	720.0	358.3	727.0	2571.6	7726.1
Skew	1.188	2.869	-0.226	3.208	2.656	2.107
% zero	26	24	38	14	2	0
10% conf.	839.0	522.0	525.0	905.0	3174.0	15416.0
90% conf.	-266.0	-429.0	-429.0	-124.0	285.0	462.0

Table E.9: Gorge Weir Historical Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	0.	0.	293.	156.	-153.	-334.
Maximum	43923.	36522.	27915.	26168.	8148.	3842.
Mean	9643.6	11716.9	8290.7	4933.3	1782.2	689.0
Median	5237.5	8332.5	4752.5	3095.0	1068.0	512.0
std. dev.	10541.5	10664.8	7672.6	5822.7	1833.0	708.5
Skew	1.668	0.827	0.866	1.991	1.898	2.047
% zero	2	2	0	0	2	12
10% conf.	23018.0	29087.0	21245.0	14019.0	3843.0	1440.0
90% conf.	484.0	1542.0	579.0	460.0	175.0	-58.0

Table E.10: Gorge Weir Historical Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-460.	-673.	-156.	-373.	-4.	293.
Maximum	1960.	2830.	29402.	3001.	17340.	******
Mean	319.7	256.1	675.2	507.0	1910.8	6352.9
Median	248.4	120.0	87.3	361.4	1106.3	1827.8
std. dev.	461.3	642.1	3269.8	628.2	2588.9	16031.2
Skew	0.970	1.391	6.392	1.411	3.200	4.672
% zero	26	41	0	20	1	0

Table E.11: Gorge Weir Generated Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	-330.	-556.	-355.	37.	-119.	-327.
Maximum	87792.	89980.	67065.	49598.	11741.	3187.
Mean	10161.8	12465.7	8820.3	5298.0	1813.0	694.2
Median	6000.2	7976.7	5620.4	2951.7	1272.1	559.6
std. dev.	13503.6	14564.2	10542.3	7393.7	1919.6	670.2
Skew	3.096	2.703	2.819	3.386	2.425	1.239
% zero	2	2	2	1	3	11

Table E.12: Gorge Weir Generated Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-167.	-145.	-160.	-1.	49.	17.
Maximum	991.	1921.	432.	953.	6571.	13587.
Mean	121.1	148.2	96.4	142.9	652.3	2390.1
Median	64.5	46.0	53.0	68.5	207.0	571.0
std. dev.	179.5	345.2	115.6	203.7	1168.7	3832.8
Skew	2.951	3.762	1.224	2.505	3.552	1.922
% zero	2	7	5	2	0	0
10% conf.	279.0	272.0	268.0	346.0	1159.0	9288.0
90% conf.	18.0	14.0	17.0	14.0	52.0	66.0

Table E.13: Gumeracha Weir Historical Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	117.	128.	110.	85.	-42.	-57.
Maximum	17441.	22871.	12950.	12477.	3521.	2128.
Mean	4460.5	6086.7	4000.7	2220.9	624.7	224.5
Median	1769.0	4680.5	2259.0	1139.5	345.0	143.5
std. dev.	4787.9	5964.4	3914.6	2769.1	713.0	356.8
Skew	1.118	1.177	0.900	1.987	2.143	3.824
% zero	0	0	0	0	5	10
10% conf.	12124.0	15661.0	10404.0	4922.0	1568.0	417.0
90% conf.	244.0	716.0	364.0	195.0	100.0	0.0

Table E.14: Gumeracha Weir Historical Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-119.	-122.	-110.	1.	45.	19.
Maximum	671.	1318.	459.	1396.	20573.	79232.
Mean	120.9	137.7	96.0	143.7	842.0	3241.3
Median	95.4	69.8	81.7	78.7	228.5	732.5
std. dev.	150.3	241.1	111.5	207.7	2479.5	9745.6
Skew	1.094	2.202	0.750	3.299	5.024	5.090
% zero	21	31	20	0	0	0

Table E.15: Gumeracha Weir Generated Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	-50.	-123.	-36.	57.	-42.	-53.
Maximum	54436.	54692.	42439.	30426.	4943.	1681.
Mean	4963.7	6599.3	4417.2	2510.2	639.0	222.9
Median	2494.3	3915.4	2441.1	1141.9	403.4	136.4
std. dev.	7882.0	8502.7	6282.4	4300.8	773.2	287.1
Skew	3.677	3.036	3.352	3.956	2.849	2.451
% zero	2	2	1	0	3	12

Table E.16: Gumeracha Weir Generated Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-1381.	-405.	-490.	-78.	114.	97.
Maximum	2859.	5927.	946.	16984.	26617.	62725.
Mean	366.7	514.8	269.8	1079.9	3543.5	8664.8
Median	310.5	271.5	231.5	524.0	1830.0	3257.5
std. dev.	663.9	1132.7	295.3	2595.2	5464.3	12919.2
Skew	1.368	3.500	0.205	5.473	2.968	2.557
% zero	14	21	14	2	0	0
10% conf.	722.0	871.0	620.0	1775.0	6364.0	22093.0
90% conf.	-233.0	-198.0	-94.0	72.0	557.0	931.0

Table E.17: Onkaparinga Historical Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	779.	1741.	953.	840.	-50.	-443.
Maximum	69879.	66025.	39687.	31054.	14061.	2844.
Mean	15857.7	18079.3	11996.8	7605.7	2794.9	826.9
Median	13393.0	15874.5	7964.5	4601.0	1366.0	708.0
std. dev.	14638.7	14387.5	10252.7	7979.4	3020.9	704.9
Skew	1.652	1.148	1.066	1.524	1.850	1.019
% zero	0	0	0	0	2	5
10% conf.	39865.0	38812.0	28863.0	22354.0	6596.0	1810.0
90% conf.	1590.0	3403.0	2254.0	1058.0	650.0	112.0

Table E.18: Onkaparinga Historical Inflow Statistics (July to December)

Flow Volumes (ML)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-953.	-358.	-415.	-51.	154.	156.
Maximum	2283.	4778.	1091.	9894.	46098.	******
Mean	374.2	491.1	267.8	971.6	3740.0	9309.7
Median	327.7	236.8	253.4	515.1	1811.8	3871.6
std. dev.	639.9	849.4	299.2	1459.7	6331.5	17169.2
Skew	0.433	2.388	0.222	3.463	3.952	3.951
% zero	30	29	19	4	0	0

Table E.19: Onkaparinga Generated Inflow Statistics (January to June)

Flow Volumes (ML)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	-111.	45.	447.	570.	4.	-381.
Maximum	******	95370.	82775.	83104.	21454.	3181.
Mean	16367.0	18695.4	12554.4	8329.6	2835.8	827.4
Median	11409.5	13980.7	8693.9	4590.7	1817.8	721.5
std. dev.	16917.9	16705.1	13011.8	11908.9	3332.7	691.9
Skew	2.512	2.020	2.663	3.661	2.910	0.904
% zero	1	1	0	0	1	8

Table E.20: Onkaparinga Generated Inflow Statistics (July to December)

RainFall (mm)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	0.	0.	3.	44.	128.	150.
Maximum	1207.	1657.	1295.	2303.	2548.	2499.
Mean	281.9	334.9	266.9	702.2	1098.6	974.3
Median	163.5	209.5	130.5	680.5	934.0	938.5
std. dev.	267.5	369.4	290.5	517.3	669.6	591.1
Skew	1.256	1.710	1.553	0.886	0.604	0.775
% zero	2	10	0	0	0	0
10% conf.	673.0	676.0	677.0	1420.0	2106.0	1951.0
90% conf.	33.0	0.0	26.0	77.0	367.0	253.0

Table E.21: Millbrook Reservoir Rainfall Historical Statistics (January to June)

RainFall (mm)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	406.	138.	185.	20.	13.	16.
Maximum	2395.	2320.	2109.	2203.	1351.	965.
Mean	1270.8	1111.5	940.7	818.6	517.2	362.6
Median	1287.0	1027.0	799.0	718.0	455.0	333.5
std. dev.	519.2	499.0	526.5	523.1	299.9	231.7
Skew	0.123	0.504	0.653	0.764	0.778	0.678
% zero	0	0	0	0	0	0
10% conf.	1982.0	1940.0	1748.0	1543.0	889.0	736.0
90% conf.	509.0	535.0	313.0	242.0	182.0	67.0

Table E.22: Millbrook Reservoir Rainfall Historical Statistics (July to December)

RainFall (mm)						
Month	Jan.	Feb.	Mar.	Apr.	May	Jun.
Minimum	-10.	-28.	-1.	-96.	88.	30.
Maximum	2279.	3152.	3133.	2965.	4204.	3416.
Mean	294.6	353.1	293.3	714.5	1118.2	977.4
Median	190.6	197.1	150.1	592.1	957.9	850.2
std. dev.	350.7	486.3	457.2	555.0	735.3	628.1
Skew	2.885	3.028	3.435	1.435	1.511	1.274
% zero	2	5	1	2	0	0

Table E.23: Millbrook Reservoir Rainfall Generated Statistics (January to June)

RainFall (mm)						
Month	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Minimum	197.	166.	125.	3.	12.	-26.
Maximum	2941.	2816.	3196.	3057.	1601.	1219.
Mean	1281.9	1126.4	953.6	824.5	514.1	361.1
Median	1232.1	1060.7	839.9	712.5	465.2	318.8
std. dev.	538.2	513.2	561.3	553.0	302.9	236.9
Skew	0.534	0.708	1.323	1.360	1.001	1.076
% zero	0	0	0	1	0	1

Table E.24: Millbrook Reservoir Rainfall Generated Statistics (July to December)

Corrigenda for

Risk and Reliability Assessment of Multiple Reservoir Water Supply Headworks Systems

by

Mr. P.D. Crawley

Page	11	In paragraph 1, line 1, insert "It is recognised that the water distribution network between the finished water storage and the users is an important part of the system and does affect the system reliability. This area of a bulk water supply system has not been considered in the work undertaken presented in this thesis." after the sentence ending ".... in greater detail in Section 2.4.".
Page	14	In line 1 replace "reliable" with "reliability".
Page	45	Delete paragraph 4 commencing "It is generally considered ....".
Page	71	In paragraph 1, line 1, replace "Attainment of the goals is sought sequentially beginning with the highest priority goal. Only when a goal is attained is any consideration given to the next lower priority goal." with "There are two types of goal programming - weighted goal programming and hierarchial goal programming. In hierarchial goal programming, the next lower goal can be considered once the previous higher priority goal is achieved to its maximum possible under the constraints, a condition that may not correspond to complete attainment of that goal. In weighted goal programming, all goals are handled simultaneously in relation to the weights assigned to them, with the resulting level of achievement of each goal reflecting its assigned weight. Depending on the nature of the constraint set, higher priority goals are not always achieved to the same extent as lower priority goals." In paragraph 1, line 7, replace "goal programming" with "weighted goal programming". In paragraph 1, line 9, rcplacc "goal programming" with "weighted goal programming".
Page	91	In paragraph 2, line 3, replace "precision" with "accuracy".
Page	102	In paragraph 3, line 2, replace "in a system **is** limited or non-existent" with "in a system **are** limited or non-existent".

Page 103 In paragraph 4, line 10, insert "The type of cue may also affect the reconstruction of events from memory. Considerable care is required in the choice of cues to avoid bias in the reconstruction of events from memory." after the sentence ending "... normal recall processes.".

Page 104 Move the heading "**Creative Activity**" at the bottom of Page 104 to the top of Page 105.

Page 118 In paragraph 2, line 8, replace "reliability indice estimates" with "estimates of the indices".

Page 124 Move the array T[*i*,*j*] to the top of Page 125.

Page 153 At the end of paragraph 5 after ".... of gathering experts around a table.", insert "These advantages include :

- Raising the level of interest and enthusiasm in the process.
- Ensuring outside interruptions are kept to a minimum.
- Ensuring the group is focussed on the problem at hand.
- Ensuring the actual site conditions and limitations are considered
- Providing additional visual stimulus regarding related components
- The potential for issues to be raised that may not otherwise have been considered.".

Page 211 At the bottom of this page insert the following paragraph. "The actual values assigned to the coefficients δ^{n}, γ_{t}^{o} and α_{t}^{n} should ideally be determined from a sensitivity analysis involving an investigation of the change in operations arising from variation to these values. This sensitivity analysis would be a time consuming task and is not considered feasible for this study."

Page 213 In the definitions under Equation 4.4 replace "Demand Zone" with "demand zone" and "Demand zone Transfer" with "Demand zone transfer".
Replace Equation 4.4 with :

$$D_{t}^{m} = F_{t}^{m} - \sum_{x=1}^{X} ZT_{t}^{mx} + \sum_{y=1}^{Y} ZT_{t}^{ym} \quad \forall\ m,t$$

Page 214 In paragraph 2, line 1, replace "input data that **is** used" with "input data that **are** used".

Page 231 In paragraph 3, line 1, replace "Inflow data **has**" with "Inflow data **have**".

Page 234 In paragraph 1, line 1, replace "Inflow data **has**" with "Inflow data **have**".

Page 240 In the definitions under Equation 4.10 replace "Sample Probability" with "Sample probability".

Page 250 In paragraph 1, line 4, replace "**this** synthetic data" with "**these** synthetic data".
In paragraph 2, line 1, replace "rainfall data **has**" with "rainfall data **have**".

In paragraph 2, line 2, replace "**This** data **has**" with "**These** data **have**".
In paragraph 2, line 9, replace "gauging data **is**" with "gauging data **are**".

Page 295 In Table 4.33, column 5, replace "Gl/Month" with "GL/Month".

Page 296 In Table 4.35, column 5, replace "Gl/Month" with "GL/Month".

Page 297 In Table 4.37, column 5, replace "Gl/Month" with "GL/Month".

Page 298 In Table 4.39, column 5, replace "Gl/Month" with "GL/Month".

Page 301 In paragraph 2, line 4, replace "**comprises**" with "**comprise**".

Page 310 In paragraph 2, line 1, replace "*Qr*" with "*Qr*".

Page 318 In paragraph 4, line 1, replace "in **section** 4.2.4.2" with "in **Section** 4.2.4.2".

Page 329 In paragraph 2, line 1, replace "demand data **has**" with "demand data **have**".

Page 345 In paragraph 2, line 8, replace "in **section** 5.3" with "in **Section** 5.3".

Page 346 In paragraph 4, line 1, replace "in **section** 4.2.4.2" with "in **Section** 4.2.4.2".

Page 352 In paragraph 2, line 1, replace "in **section** 4.2.4.1" with "in **Section** 4.2.4.1".
In paragraph 4, line 2, replace "system **has** been" with "system **have** been".
In paragraph 5, line 1, replace "demand data **has**" with "demand data **have**".

Page 362 In paragraph 3, line 5, replace "**this** data" with "**these** data".

Page 365 In paragraph 3, line 1, replace "synthetic data **has** " with "synthetic data **have**".
In paragraph 3, line 2, replace "**this** data **has**" with "**these** data **have**".

Page 374 In paragraph 4, line 1, replace "demand data **has** " with "demand data **have**".
In paragraph 4, line 2, replace "failure data **has** " with "failure data **have**".

Page 385 In paragraph 2, line 1, replace "demand data **has** " with "demand data **have**".
In paragraph 2, line 2, replace "failure data **has** " with "failure data **have**".

Page 391 In paragraph 1, line 6, replace "reservoir" with "reservoirs".
Delete paragraph 3 commencing "A synthetic inflow ....".

Page 394 In paragraph 1, line 2, after "impact" insert "of bulk water transfer failures".

Page 413 In Table 5.60, column 4, replace "$M/Gl" with "$M/GL".

Page 457 In paragraph 1, line 1, replace "in **section** 5.3" with "in **Section** 5.3".
In paragraph 2, line 2, replace "in **section** 5.2" with "in **Section** 5.2".

Page	460	In paragraph 1, line 7, replace "souther" with "southern".
Page	467	In paragraph 3, line 5, replace "authors" with "author's".
Page	468	In paragraph 3, line 1, replace "application of the methodology" with "application of a methodology". In paragraph 3, line 4, remove "with little or no reduction in system reliability".